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An Investigation of Monotonic and Cyclic Behaviour of Leda Clay

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AN INVESTIGATION OF MONOTONIC AND CYCLIC BEHAVIOUR OF LEDA CLAY

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Abstract

The behaviour of a sensitive clay from Ottawa is investigated though strain- and stress-controlled cyclic simple shear, and resonant column tests. Strain-controlled tests were performed at frequencies 0.1 to 10Hz, and amplitudes 0.1% to 5%. Over this range of strain amplitudes there are large drops in stiffness. Strain amplitude has a greater effect on the generation of pore-pressures than frequency. Stress-controlled tests performed at a yield stress ratio of 1 had higher strength at low N but degraded more readily than a test at YSR =2. A novel method of remoulding clay and measuring the remoulding energy is also presented. Specimens of remoulded and intact clay are used in fall cone, and oedometer tests in order to quantify sensitivity, and the breakdown of structure. Results suggest that the sensitivity framework (Cotecchia and Chandler 2000) and the relationship between PI and γ_t (Hsu and Vucetic 2006) are applicable to this clay.

Keywords: Cyclic behaviour, sensitive clay, remoulding, strain rate, frequency, structure, Leda clay, Champlain Sea clay, clay.

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S_u – Undrained shear strength

τ_c – Cyclic shear strength

τ_f – Cyclic shear strength at failure (3% strain)

N – Number of uniform cycles

N_f - Number of uniform cycles at failure (3% strain)

σ'_{vo} – Initial effective vertical stress

σ'_{vy} – Effective vertical yield stress

σ'_v – Vertical effective stress

γ_a – Cyclic strain amplitude

γ_{sf} – Static failure strain

γ_a / γ_{sf} - Cyclic strain ratio, cyclic strain amp/ static failure strain

γ_t – Threshold shear strain for cyclic pore-water pressure

e - Void ratio

I_v - Void index

e^*_x - Intrinsic void ratio under a vertical effective stress of 'x' kPa

e_L - Void ratio at the liquid limit

S_t - Sensitivity

G - Shear modulus

G_o - Initial small-strain shear modulus

G_u - Shear modulus of an undisturbed or intact specimen

G_d - Shear modulus of a disturbed or remoulded specimen

f - Frequency

α - Slope of regression line from Khan et al. 2010

S_{ta} - Apparent sensitivity

U - Shear strength after complete remoulding in mechanical mixer

A - Shear strength after 15 revolutions of a standard lab vane

w_c - Natural water content

LL - Liquid Limit

PL - Plastic Limit

PI - Plasticity Index

LI - Liquidity Index

G_s - Specific Gravity

R_u - Excess pore-water pressure ratio

Chapter 1

1 Introduction

Clay soil that is characterized by undrained shear strength in the remoulded state that is much lower than its undrained shear strength in the undisturbed state at the same water content is classified as sensitive. Sensitive clays cover large areas of many northern countries that were once covered by glaciers such as Canada, Norway and Sweden. In Canada, sensitive clays underlie one of the most densely populated areas in Canada and present a significant potential hazard. Large flow-slides in sensitive clay have unexpectedly occurred with sometimes fatal consequences (Tavenas et al., 1971; CBC News). It is therefore necessary to better understand the response characteristics of sensitive clay to static and cyclic loads and accurately predict its seismic response in order to facilitate reliable design practices for foundations and buildings situated on sensitive clays.

There have been various definitions of what constitutes a clay of 'Low' or 'High' sensitivity beginning with Skempton and Northey (1952) who based their classification on data collected from tests on Norwegian clays. From 1952 onward, there have been different classifications from multiple researchers and countries. Most recently in Canada, the Canadian Foundation Engineering Manual (CFEM, 2006) was issued with one definition closely resembling the Swedish classification (2004) and then later with an errata that more closely resembled the classification of Skempton and Northey (1952). The various classification systems available point to the variability of sensitivity test results and the variability of sensitivity clay properties between regions.

In situ and laboratory tests can be used to determine sensitivity. The Field Vane Test and Cone Penetration Test are the most common tests used in the field and the Unconfined Compression Test, the Lab Vane Test, and the Fall Cone Test are the most common in the lab. It has been shown that different tests will give different values of sensitivity (Eden and Kubota, 1961). Varying results have been attributed to insufficient accuracy in an apparatus for measuring very low remoulded strengths, time lapse between remoulding the clay and testing it, whether the test is conducted with ascending or descending rates of shear. Differing methods and degrees of remoulding also contribute to variation in sensitivity values (Abuhajar et al., 2010).

The objective of this research is to better understand the behaviour of sensitive clays from the Ottawa area (Leda clay) and to generate relationships between properties that are useful for design. This study will examine the properties and strength behaviour of Leda clay, including evaluation of: Atterberg limits, specific gravity, and grain size distribution using the appropriate ASTM tests. An apparatus that is able to measure energy during remoulding is developed and employed to produce remoulded specimens of sensitive clay. Oedometer tests are used to determine the compressibility behaviour of both remoulded and undisturbed specimens. Static and cyclic simple shear tests are carried out in order to determine the undrained strength behaviour. The remoulded specimens are then subjected to 1-dimensional consolidation in order to determine the stress sensitivity (Cotecchia and Chandler, 2000).

Stress and strain controlled cyclic tests are used to find the degradation of sensitive clay stiffness with cyclic strength ratio, number of cycles, frequency, and strain amplitude. In addition, a resonant column test is conducted on a sensitive clay specimen in order to provide data on its small strain properties. An alternate method of remoulding is presented and the sensitivity is evaluated using the fall cone in order to compare with other techniques for measuring clay sensitivity. The results from these tests can be used to inform future designs on sensitive clay from the Ottawa area.

1.1 Organization of Thesis

First, a review of the origin and development of sensitive clays in eastern Canada is presented. Knowing the geological history of the clay and how it came to be sensitive is important for understanding its behaviour. Next, studies on the behaviour of Leda and Champlain sea clay are reviewed. The relevant information as it pertains to this study of previous research on the compressibility, monotonic and cyclic shear strength are also included. Monotonic and cyclic simple shear tests are used to determine the behaviour of Leda clay in this study and so the appropriateness of using this device is assessed through review of studies on different important aspects, including: the distribution of stresses within a clay sample in simple shear, its equivalence to a truly undrained test, and the height to diameter ratio of the specimen. Definitions of what constitutes a clay of ‘low’ or ‘high’ sensitivity are different between many published classifications and those classifications are reviewed. Finally, methods of remoulding sensitive clay that are used to find the sensitivity are reviewed.

Chapter 3 describes the tests undertaken in this study. The tests performed included Atterberg limits, specific gravity, and hydrometer in order to determine the general properties of the clay; oedometer to determine the yield stress and compressibility behaviour; remoulding and fall cone tests to determine sensitivity; monotonic and cyclic simple shear tests; and resonant column tests. A new remoulding apparatus and method are also introduced in this chapter.

The results of the aforementioned tests are presented and discussed in Chapter 4. A comparison between the results obtained in this study and published results in the literature, and discussion of how the present studies results fit into published works such as Burland’s (1990) intrinsic compressibility, Vucetic’s (1994) threshold shear strain, Vucetic and Dobry’s (1991) shear modulus degradation curves, and cyclic strength ratio vs. number of cycles for a Champlain Sea clay (Lefebvre and Pfendler, 1996). The effect of strain rate on shear strength is also investigated and compared to the strain rate effect of similar clays.

In Chapter 5, conclusions are drawn from the results of this study and how they fit into previous studies. The need for additional research is also discussed.

Chapter 2

2 Literature Review

2.1 The Origin and Development of Sensitive clays

Clays with higher undisturbed undrained shear strengths than remoulded strengths are considered sensitive. Sensitive clays are found around the world but prominently in locations that were once glaciated. In Canada, one of the largest deposits of sensitive clay can be found along the St. Lawrence River and the Ottawa valley in the footprint of the historic Champlain Sea. The Champlain Sea existed approximately 12,500 - 10,000 years BP at the end of the last ice age as shown in Figure 2.1. The retreating ice allowed water from the Atlantic Ocean to flow into the St. Lawrence and Ottawa valleys where fine sediments were deposited over a long period of time. Isostatic uplift followed glacial retreat, draining the sea and exposing the clay.

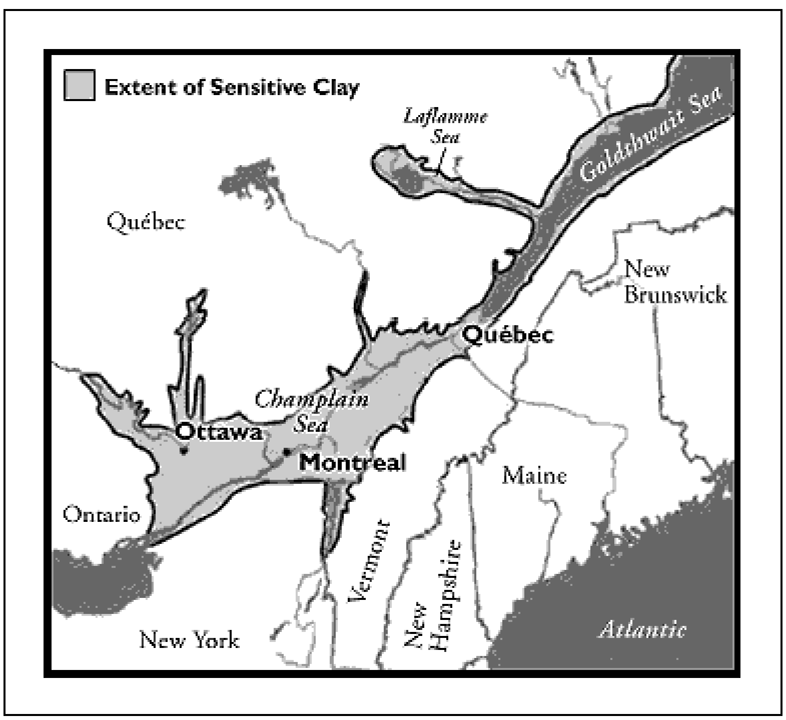


Figure 2.1: Extent of sensitive clay in Eastern Canada (Canadian Mortgage and Housing Corporation, 2001).

The clay was partially derived from glacial sediments produced by the grinding of the unweathered rock of the Canadian Shield by the heavy ice sheets. The rock flour of some Eastern Canadian sensitive clays is made up of plagioclast, quartz, microcline, horneblende, dolomite and calcite (Locat et al., 1984) in decreasing order of abundance as shown in Table 2.1. A number of specimens were tested from each sample as denoted by ‘N’. The St. Marcel, St. Léon, St. Alban, St. Barnabé and Shawinigan clay samples are from the Champlain Sea basin. In Table 2.1, ‘P’ represents the percentage of phyllosilicates and amorphous materials as the arithmetic difference between the total dry weight of the sample and the dry weight of the six major minerals. The amount of major minerals in the clay fraction is also given by the difference between the clay fraction and ‘P’. Organic content is assumed to be negligible.

Table 2.1: Mineralogy of Eastern Canadian Sensitive clays (Locat et al., 1984). Averages are shown as decimals while the range of results is shown in parentheses.

Site	N	Plagioclast (%)	Quartz (%)	Microcline (%)	Hornblende (%)	Dolomite (%)	Calcite (%)	P (%)	Illite (%)
Grande-Baleine	4	41.3 (39-46)	15.3 (14-16)	13.5 (13-14)	12.7 (11-15)	1 (0-2)	0.3 (0-0.5)	15.9	7.5
Grande-Baleine	5	47.9 (41-57)	13.9 (13-16)	13.8 (12-16)	11.8 (11-13)	3.5 (1-5)	0.6 (0.3-0.9)	8.5	7.2
Grande-Baleine	5	40.4 (37-48)	16.8 (13-20)	14.5 (13-16)	11 (9-14)	1.9 (1-3)	0.4 (0.2-0.6)	15	5.7
Olga	6	29.1 (22-34)	11.4 (10-13)	9.6 (8-11)	9.4 (7-10)	3.7 (0-5)	2.3 (1.5-2.7)	34.5	10.2
Olga	3	33.1 (31-36)	13 (11-15)	10.8 (8-13)	8.6 (7-9)	2.5 (2-3)	0.7 (0.5-0.9)	31.3	9.4
St. Marcel	3	35.6 (35-36)	11 (11)	8.9 (8-10)	11.4 (9-14)	5.2 (3-8)	1.2 (1.0-1.6)	26.7	9.2
St. Marcel	3	33.7 (34)	13.9 (13-14)	9.8 (8-11)	11.1 (9-12)	5 (5-6)	1.6 (1.5-1.7)	24.9	8.9
St. Léon	15	36.3 (27-45)	11.8 (10-13)	13.4 (11-16)	12.6 (10-16)	3.6 (2-5)	0.1 (0-0.4)	32.2	11.1
St. Alban	19	25.1 (22-33)	20.5 (17-24)	9.5 (8-13)	9.5 (5-14)	1.5 (0-5)	0.7 (0-1.2)	33.2	11.2
St. Barnabé	3	37 (33-40)	12.2 (11-13)	14.4 (12-16)	13.3 (12-15)	3.5 (3-5)	0.2 (0.2-0.3)	19.4	6.9
Shawinigan	5	36.8 (32-42)	19.6 (17-24)	15 (14-16)	13.4 (12-17)	2.4 (0-4)	-	12.8	9.4
Chicoutimi	8	41.1 (35-44)	19.8 (17-23)	13.2 (12-16)	9.7 (7-13)	1.5 (0-3)	0.7 (0.4-1.0)	14	2.7

Sensitive clays in Canada have been found to contain all of these minerals but to varying degrees in different locations. The grinding process does not usually produce significant amounts of active clay minerals and the activity number of the clay is often less than 0.5 (Mitchell and Soga, 2005), where activity number is defined as plasticity index divided by the percent clay sized particles less than 2 μm in diameter. Plasticity index of sensitive clay can vary between 4 and 40 and the clay-size fraction (<0.002 mm) between 80% in the lowlands and 30% at the historical shore of the Champlain Sea (Mitchell and

Klugman, 1979). Sensitive clays are also characterized by a Liquidity Index of 1 or higher. Cementation bonds are believed to allow the clay to maintain such a high water content. Biochemical or chemical change such as leaching may have also decreased the liquid limit to a level lower than the natural water content (Lefebvre, 1984).

It is thought that sensitivity is caused by the presence of a metastable fabric, nature of cementation between clay particles, weathering, thixotropic hardening, leaching or changes in the pore water chemistry, and ageing. The sensitivity of clay may be attributed to more than one mechanism. Some important features of the depositional and post-depositional environments needed to produce quick clay are shown in Figure 2.2 (Torrance, 1983 modified from Quigley, 1980).

Factors affecting sensitivity	
Factors that contribute to a high undisturbed strength	
Depositional	**Post-depositional**
Flocculation	Cementation bonds
salinity divalent cations adsorption high suspension concentration	rapidly developed slowly developed
	Slow load increase time for:
	cementation thixotropic processes
Factors that contribute to a low remoulded strength	
Depositional	**Post-depositional**
Material properties	Minimal consolidation
low activity minerals dominate	
	Leaching
	decrease in LL > decrease in w_c
	Dispersants
	decrease in LL > decrease in w_c

Figure 2.2: Factors of the depositional and post-depositional environments that affect sensitivity (after: Torrance, 1983 modified from Quigley, 1980).

Flocculated particles in an open 'house of cards' type arrangement, make up the fabric of highly sensitive clays (Figure 2.3). An open fabric is a necessary feature of highly sensitive soils but not necessarily for low sensitivity clay. Two soils can have the same fabric but different forces between particles and particle groups, which will give them different properties. The soil structure is a function of the fabric and its stability, which is sensitive to changes in stress and chemical environment. Highly sensitive clays may not differ from clays with low sensitivity in terms of mineral composition, grain size distribution or fabric but because of the interparticle forces and the chemical environment present, a clay can be sensitive (Mitchell and Soga, 2003).

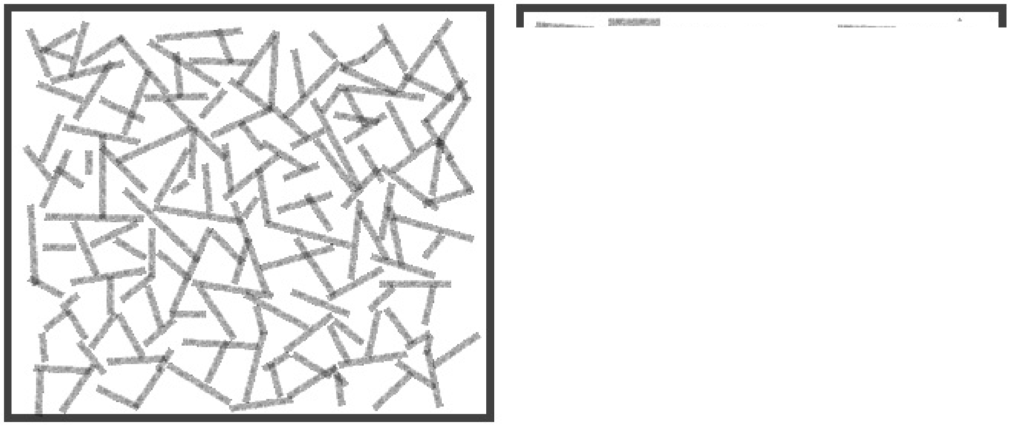


Figure 2.3: a) Undisturbed structure and b) disturbed, collapsed structure of highly sensitive clays.

The nature of particle cementation is thought to be a cause of high sensitivity in Canadian clays. Many researchers that have studied Leda clay behaviour (e.g. Kenney et al., 1967; Walker and Raymond, 1968; Townsend et al., 1969; Moum and Zimmie, 1972; and Sangrey, 1972a, 1972b) considered it to be cemented. Particles of carbonates, iron oxide, alumina and organic matter may precipitate at interparticle contacts and act as cementing agents, giving the clay an increased strength. The yield stress will also be influenced by the degree of cementation.

The distribution of deposition, stressing, and bonding rates, as well as mineralogy and pore-water chemistry contribute to the final structure of the soil, over a long period of time (e.g., Quigley and Ogunbadejo, 1972b; Bentley, 1980; Quigley, 1980).

Leaching can change the chemistry of clay deposits and hence leads to sensitive clay behavior. In order to determine the effect of certain cations on sensitivity, Loiselle et al. (1971) leached Fe^{3+} and Ca^{2+} compounds from Quebec Champlain clays using EDTA. With leaching, a reduction in vertical yield pressure occurred; however, EDTA leaching can also remove carbonates. Boon and Lutenegger (1997) suggest that due to the open fabric of the clay, there are fewer particle contacts where cementation bonds can form and therefore carbonates have a more limited effect on the vertical yield pressure. It was also reported that S_u of Quebec clays reduced by 10–20% under sustained loads through the removal of 0.43% of Fe 3+ and 0.22% of Ca 2+ compounds, by dry weight.

Weathering can occur through mechanical and chemical means. The weathered crust of a Leda clay deposit consists of an upper oxidized layer containing fissures in the region of seasonal groundwater fluctuation and a layer below of grey clay. The crust thickness varies but is usually within a depth of 10 m and can be identified by a decrease in field vane strength with an increase in depth (Eden and Mitchell, 1970). Weathering can also change the contents of the pore water. The type and proportions of ions in the pore water affect the flocculation and deflocculation of the soil when remoulded and therefore the remoulded strength.

Thixotropy is an isothermal, reversible and time-dependent process that occurs under a constant composition and volume where by a material stiffens at rest and becomes liquid upon remoulding. Thixotropic hardening may account for low to medium sensitivities (up to 8 or so) and for part of the sensitivity of quick clay (Skempton and Northey, 1952).

Leda and Champlain Sea clays were deposited in brackish environments and retained their saline pore water until glacial retreat caused geostatic uplift and fresh water began to infiltrate the clay via sand and silt lenses. The fresh water did not necessarily flow through the clay but removed the salt through diffusion (Torrance, 1974). Rosenqvist (1946) studied the influence of pore water salinity on sensitivity and found that for Norwegian sensitive clays sensitivity was significantly influenced by the amount of salt in the pore water. Leaching does not affect the sensitivity by changing the clay fabric rather it changes the interparticle forces. Penner (1965) found the electrokinetic potential of Leda clay to correlate well with sensitivity. The electrokinetic potential is a measure of the double layer potential of a particle; the less electrolytes in solution, the thicker the double layer, the higher the repulsive forces, the higher the sensitivity. The sensitivity of Champlain clay does not necessarily depend as greatly on salt content. Though the clay rarely has a salt content over 1 - 2 g/L, very high sensitivities are still present (10 – 1000 (Eden and Crawford, 1957; Penner, 1963c, 1964, 1965)). This shows that though leaching of salt is necessary to develop a quick clay, it may not be enough in all cases.

The most important condition in order for clay to behave as quick clay is an increase in interparticle repulsion forces. The type and relative amounts of mono and divalent cations control the equilibrium particle arrangements and the increase in interparticle repulsion resulting in an increase in sensitivity. For low salinity clay like Champlain Sea clay, sensitivity correlates well with the percent of monovalent cations (Moum et al., 1971). According to data by Penner (1965), the relationship between sensitivity and percent of monovalent cations, the select removal of divalent cations is necessary for high sensitivities to develop. Calcium and Magnesium ions are removed possibly by organic matter that is deposited simultaneously with the illite, feldspar, and quartz that make up to bulk of postglacial marine clay (Soderblom, 1969; Lessard and Mitchell, 1985).

Ageing can also contribute to the sensitivity of a soil. By minimizing the exposure of a sample to air, using rust free shelby tubes, and storing samples at low temperatures to slow reaction rates can prevent the changing of properties due to ageing. By comparing the pH of the sample before storage and after use can indicate how the properties have changed since sampling (Mitchell and Soga, 1993).

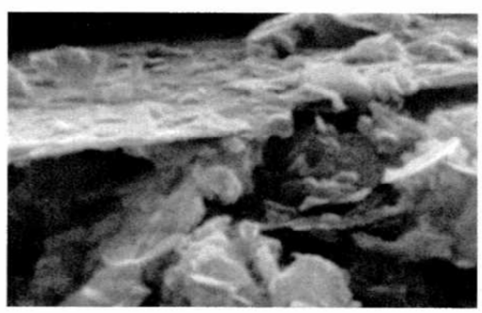


Figure 2.4: Undisturbed Leda Clay. Photo is 8micrometers wide (Tovey, 1971).

2.2 The Behaviour of Sensitive Clays

2.2.1 Compressibility

In general, a plot of a 1-dimensional consolidation test of Champlain Sea (sensitive) clay has a noticeable break at the yield stress where it becomes very compressible and a progressive decrease in slope with larger stresses. The abrupt break has been associated with failure of cementation bonds (Mitchell, 1970) and the progressive collapse of inter aggregate pores (Delage and Lefebvre, 1984; Lapierre et al., 1990). The position of compression curve in e-log σ'_v space is also determined by bonding between aggregates (Delage, 2010). Though the clay is geologically normally consolidated, bonding and microstructure cause the clay to yield at loads higher than the in-situ vertical stress.

Burland (1990) presented a framework for interpreting the properties of natural clays based on observations from an experimental investigation that involved natural and reconstituted clays. A clay specimen that has been thoroughly mixed at 1 to 1.5 times its liquid limit is considered reconstituted. To attain a reconstituted state, the clay should not be air or oven dried and ideally the pore water chemistry is kept similar to a natural specimen.

In 1-dimensional consolidation, a plot of void ratio vs. vertical pressure gives a range of curves for different clays. The void ratios at 100 and 1000 kPa for a certain clay are termed the intrinsic void ratios for that clay and the difference between them is termed the intrinsic compression index; these terms can be used to find the void index, I_v. The void index is a measure of the intrinsic compactness of a soil and is given by

$$\boldsymbol{I_v = \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}}} \qquad \mathbf{2.1}$$

where * indicates an intrinsic state. Using the void index, the original void ratio vs. vertical pressure curves can be normalized in order to achieve a reasonably unique line: the 'intrinsic compression line' (ICL).

The ICL is used to characterize the undisturbed state of the soil, its mechanical properties, and structure with respect to its intrinsic (or structure-less) state. It is also useful to compare the sedimentation compression curve (Terzaghi, 1941) with the ICL, and undisturbed compression curve. Sedimentation curves can also be normalized and plotted as void index vs. vertical pressure (the sedimentation compression line, SCL). The normalized undisturbed clay compression curve plots in relation to the ICL and SCL can provide relative information on the deposition conditions of a sediment. The location of the SCL in relation to the ICL shows that for the same void index, the vertical pressured carried by the natural clay is much higher than for the reconstituted specimen. Not all the clays that Burland investigated plotted on the SCL so it is not a fundamental line. However, he maintained that it is still valid for most natural sedimentary clays. The ratio between the yield pressure of an undisturbed compression curve and the ICL at the same I_v is termed the stress sensitivity (Cotecchia and Chandler, 2000). The stress sensitivity has been shown to be approximately equal to the strength sensitivity (S_t found by field vane, fall cone, etc.).

2.3 Testing Methods for Soil Shear Strength Characterization

There are different testing methods that are used to characterize the shear strength of soils. The static behaviour of soils varies with test type, strain rate, and degree of consolidation. The most used testing methods in this study are discussed below.

2.3.1 Simple Shear

The cyclic simple shear test reproduces earthquake conditions more accurately than the cyclic triaxial test (Kramer, 1996). Subjected to stresses on top and bottom similar to vertically propagating S-waves on a soil element, simple shear tests allow the principal stress and/or strain axes to rotate while the specimen is kept under a condition of plane strain. In real engineering situations and most in-situ tests, the principal axes also rotate. Most apparatus only give information about the shear stresses on the horizontal plane and not on the vertical sides of the specimen. Since the machine only applies shear stresses to the top and bottom of the specimen, and there are no complimentary shear stresses on the sides of the specimen, there must be nonuniform shear and normal stresses present to balance them. For relatively soft clays, the compliance in the test apparatus is insignificant but for dense sands it must be considered (Andersen et al., 1980).

There have been various studies done to identify the nature of the nonuniform stresses in direct simple shear (DSS) tests. A mathematical analysis of the Cambridge simple-shear apparatus on an isotropic, linear elastic material by Roscoe (1953) showed that tension develops in the upper leading edge and the lower trailing edge. If a suitably large vertical load is applied to the specimen the tension zones do not develop but nonuniform loading is still present. Across the middle third of the specimen faces, the results showed that the shear stress is approximately uniform. Duncan and Dunlop (1969) performed a finite element analysis considering a nonlinear, anisotropic material within the Cambridge simple-shear apparatus. They found that for horizontal equilibrium to occur, the shear stress varies throughout the specimen height. The difference between the average values of shear stress at the top and at mid height ranged from 4% to 8%, which encompasses Roscoe's value of 7%.

Performing a number of Ck_oU strain-controlled tests using the Geonor DSS device, Vucetic and Lacasse (1982) investigated the influence of height to diameter (H/D) ratio on the behaviour of Haga clay at various OCRs and three H/D ratios. They found that the H/D ratio does not significantly affect the measured strength of Haga clay. Vucetic and Lacasse (1984) also found that between 50 cm^2 and 20 cm^2 specimens of Drammen clay there were minimal differences between the stress-strain behaviour of each specimen.

Constant volume tests in a Cambridge simple shear apparatus that was instrumented to measure normal and shear stresses at various locations on the top and bottom faces of the specimen demonstrated that the shear strain distribution at 10% and 20% was fairly uniform. When compared to results from the same tests on sand, clays fared much better with respect to uniformity of boundary stresses and internal deformations, due to yielding of the clay during shearing (Airey et al., 1985). DeGroot et al. (1994) found that at large strains, there is a reduction in shear resistance of a specimen due to the influence of nonuniform stress conditions on the reaction of the simple shear apparatus. At low strains, such as those at peak strength, the effect of the apparatus on the measured strength is minimal.

Bjerrum and Landva (1966) demonstrated by measuring the pore water pressure at the bottom of a specimen of Manglerud clay that, in a constant-volume DSS shear test, the change in vertical pressure is equal to the change in pore water pressure in an undrained test. Dyvik et al. (1987) investigated the equivalence of constant-volume and truly undrained tests. Tests on clay and other soils in which the pore water pressure was measured and compared to the vertical pressure change during testing confirmed what Bjerrum and Landva had found earlier, due to the constant volume condition, the change in vertical stress during cyclic shearing was considered equivalent to the change in pore pressure that would have developed in a truly undrained test.

2.3.2 Monotonic Shear Tests

In conventional triaxial tests, 1% distortional strain was found to be sufficient to induce static failure of cemented sensitive clays by many researchers (Mitchell and King, 1977). Yong and Tang (1983) statically sheared Champlain clay specimens of constant-volume undrained simple shear. They tested three samples at 74.6 kPa with a yield stress ratio (YSR) of 1.9 at a strain rate of 2.4% per minute on average. The tests showed a peak and then a steep drop which then leveled at a fairly constant value of shear stress. The strain at peak failure varied from 4% to 6%. Lefebvre and Pfendler (1996) conducted three static undrained, constant-volume tests at pressures varying from 37.8 to 45 kPa (YSR of 2.2) at a strain rate of 2.1% per hour. The shapes of the curves were similar to Yong and Tang's but the strain at failure occurred at 2.6 %. Silvestri et al. (1989) carried out anisotropic testing in the simple shear on two grey, Champlain clays. Tests showed that specimens with an undisturbed structure had an undrained shear strength that was slightly anisotropic. At pressures past yield, there was a unique relationship between effective vertical pressure and undrained shear strength and the clay did not behave anisotropically.

The static and cyclic triaxial behaviour of structured and de-structured marine and lacustrine sensitive clay from Eastern Canada was investigated by Lefebvre and LeBoeuf (1987). The monotonic tests were performed on Grand Baline, Olga, and B-6 (from the ancient Tyrrell Sea) at strain rates varying from 0.5% - 132% per hour. Grande Baleine and Olga clays were tested under isotropic conditions while B-6 specimens were anisotropically consolidated at a K_o of 0.55. The structured clays, consolidated at the in-situ overburden pressure, showed brittle behaviour and failed at an axial strain of 1% or less which is comparable to the results of Mitchell and King (1977). The peak deviator stress increased with increasing strain rate. Generation of pore water pressure up to failure was not significantly different between tests but after failure a lower strain rate would result in higher pore water pressures.

The strain rate effect in the structured marine clays seems to be related to a lowering of the failure envelope due to fatigue rather than the generation of higher pore water pressures but in the varved, lacustrine Olga clay, silt layers allow the generation of higher pore pressures at lower strain rates. In tests of clays compressed past the yield stress (destructured), pore pressure generation was significantly higher at lower strain rates, even from the very beginning of compression. The destructured clays at high strains converge to a more or less unique failure envelope. The strain rate effect in both conditions was approximately a 7-14% change in S_u/σ'_{vo} or S_u/σ'_p over the log cycle of 1-10% strain per hour. Graham et al. (1983) found an increase of 9-19% over a 0.1-1% per hour log cycle of for tests on a variety of clays.

2.3.3 Cyclic Shear Tests

Many researchers have investigated the cyclic behaviour of soils under a variety of conditions: stress- or strain-controlled; with or without an initial static bias; drained or undrained; from saturated to dry; normally consolidated or overconsolidated; and anisotropic and isotropic. Often cyclic tests are done to determine the degradation of shear stiffness or strength with varying factors such as frequency of loading, strain (or stress) amplitude, and number of cycles. Pore-water pressure is generated in undrained tests and is affected, at large enough strains, by testing conditions and loading parameters.

Mitchell and King (1977) conducted cyclic consolidated undrained triaxial tests on samples of weathered and unweathered Champlain Sea clay. The samples were consolidated isotropically to multiple different confining pressures between 25 kPa and 150 kPa. For each confining pressure different specimens were cycled at 70% of the static deviatoric stress. Another series of tests were carried out at 50 kPa with loads from 0 to 50%, 60%, 70%, 80%, and 90% of the static deviatoric stress. Two different frequencies were used: 2 and 15 cycles/min (0.033 Hz and 0.25 Hz), which were considered representative of long duration wind and wave loads. It was found that there was a large difference in the behaviour between weathered and unweathered samples at the two different frequencies. There was not a large difference at the two frequencies for unweathered specimens. The rate of pore water pressure increase was found to depend on the initial stress state, cyclic stress level and the magnitude of the stress increment (cycling between 30% and 70% vs. between 0 and 70%). In another series of tests, specimens were cycled at stress increments of 25%-55% with 70% of the static strength as an upper bound at confining pressured between 25 - 150 kPa. It was found that as long as the stress increment did not exceed 50% and the max stress did not exceed 70%, the specimens reached an ‘elastic equilibrium’ after 2000 cycles and less than 1% distortional strain. All samples from all testing series that exceeded 1% distortional strain during cyclic loading ended up failing. For some failed samples at low confining pressures, the pore water pressure decreased just before failure which has been attributed to dilation due to closely spaced planes of weakness within the soil (Eden and Mitchell, 1970). It was noticed that higher confining stresses inhibited dilation. The cyclic specimens failed on the static effective strength envelope therefore they did not fail because cyclic loading changed the effective strength envelope but because cyclic loading reduced the effective stress (increase in pore water pressure).

The behaviour of structured and de-structured sensitive clay specimens were examined by undrained, strain-controlled and stress-controlled cyclic triaxial tests (Lefebvre and LeBoeuf, 1987). Cyclic tests were performed on Grande Baleine (GB) clay at a strain rate of 1% per hour (strain-controlled) and on B-6 clay at 0.1 Hz (stress-controlled). In the tests on structured GB clay very small pore pressures were produced though the specimen still reached failure, while for de-structured tests large pore pressures were generated which brought the stress path to failure just past the static maximum deviator envelope. Structured tests with low rates of static strain and those subjected to cyclic loading exhibited the same behaviour; a lowering of the peak strength envelope.

Cyclic DSS has also been used to investigate the cyclic behaviour of Canadian sensitive clays. Lefebvre and Pfendler (1996) tested clay at the in-situ overburden pressure (a yield stress ratio of 2.2) and used stress-controlled tests at a frequency of 0.1 Hz. The monotonic tests (used to normalize cyclic strengths) were done at a strain rate of 2.1% per hour. Undrained, static, shear stress was introduced in some specimens prior to cycling, at the same shear rate as the monotonic tests. Tests with no initial static shear stress were conducted at cyclic stress ratios (τ_c/S_u) ranging from 0.64 to 1.32. Static shear stresses of 31 to 80% of the monotonic strength were applied to pre-shear tests which were done at τ_c/S_u of 0.39 to 1.23. For all tests, failure was generally at 3% to 5% shear strain.

Strain rate effects were found to be such that if the results for tests with no bias are extrapolated to failure at 1 cycle, the cyclic strength is 1.42 times the monotonic strength. Comparing the strain rates of the monotonic and cyclic test with a CSR of 1.42, the strain rate of the cyclic test is about 2000 times larger. This translates into a strength increase of about 12% per log cycle of strain which is comparable to values found by other researchers from triaxial tests (Lefebvre and LeBoeuf, 1987; Graham et al., 1983; Vaid et al., 1979). Compared to tests with initial static shear stress, the tests with no bias degraded more rapidly though the strain rate effect compensated for the initial drop in strength to 12 cycles. As the static bias increased, stress reversal decreased and became nonexistent when the static stress was larger than the cyclic stress. At 12 cycles, biased tests with stresses 0.3 to 0.8 times the monotonic strength had a 30% increase in total strength over and above the strain rate effect.

Below a certain threshold of shear strain amplitude and for the number of cycles typical of earthquake ground response, a soil's stiffness will be negligibly affected (Vucetic, 1994). There are varying, but similar, definitions of threshold shear strain. For fully saturated clays, Anderson and Richart (1976) suggested that the threshold is the shear strain that will cause a progressive decrease in secant stiffness. Vucetic (1994) defined the volumetric threshold shear strain amplitude, γ_t, as the strain below which negligible permanent pore pressures or volume changes take place. Using published data from sands and clays, Vucetic (1994) found that γ_t for clay is larger than the threshold for sands and that it generally increases with an increase in the plasticity index (PI) of the clay. This was also confirmed by Hsu and Vucetic (2006). Partially saturated soils have larger threshold strain amplitudes than fully saturated ones.

Hsu and Vucetic (2006) produced a plot of threshold shear strains versus PI, as shown in Figure 2.5. They stated that the range for a particular PI is significant means that γ_t may not be very sensitive to PI or that more data is needed to define the relationship. The limited data available on silts and clays suggests that there is not a significant effect of vertical consolidation pressure on the value of γ_t.

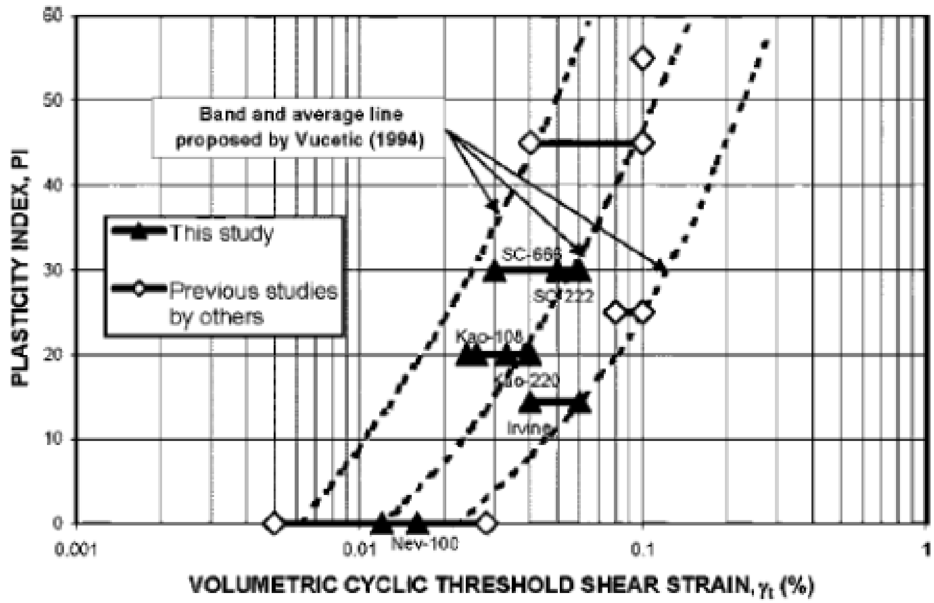


Figure 2.5: Range of volumetric cyclic threshold shear strain for a certain PI for all soils (after: Hsu and Vucetic, 2006).

Darendeli (2001) investigated the response of a variety of soils to dynamic loading at shear strain amplitudes of less than 1%. Resonant column and torsional shear tests were conducted to find the small strain frequency effect on stiffness and damping of sandy lean clay. For frequencies from 1 to 100 Hz and a constant confining pressure, there was a 10% increase in G_{max} per order of magnitude. D min instead increased markedly after 1 Hz to a 100 % increase per log cycle. Normalized G/G max curves of the same clay were not very sensitive to the frequency. The effect of frequency during cyclic loading on a large variety of soils is shown in Figure 2.6 (Stokoe and Santamarina, 2000). It can be seen that the frequency applied to a soil affects small-strain (where behaviour is linear for a given frequency) damping more than stiffness. The PI also affects the degree to which the soil responds to different frequencies. Resonant column (RC) tests on Kaolin clay using the non-resonant method gave a maximum increase of 17% in G and 16% in damping within a 1.5 to 50 Hz frequency range (Khan et al., 2010) at a confining pressure of 33 kPa and strain amplitudes less than 10^{-6}. Over 1.5 - 50 Hz there was an approximate increase in G of the Kaolin of 0.35 MPa/Hz.

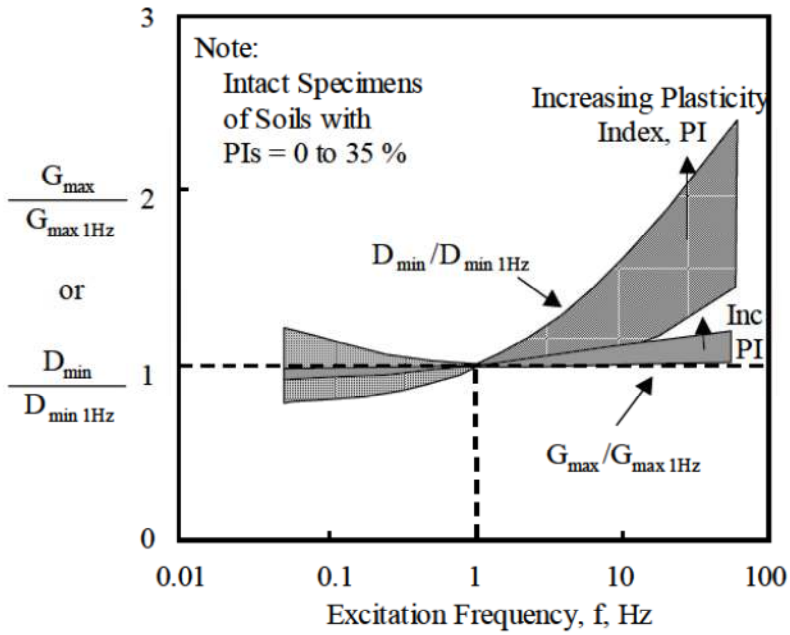


Figure 2.6: Effect of excitation frequency on small-strain stiffness and damping response (after: Stokoe and Santamarina, 2000).

Khan et al. (2010) also developed a method to reconcile conventional resonant column tests where shear modulus is found at high frequencies (35 - 75Hz) with cyclic triaxial tests performed at frequencies typically less than 2 Hz. In this method, a straight line is fitted to the results of RC tests using the non-resonant (NR) method plotted over a large range of frequencies (1-43Hz) versus the normalized shear modulus. The slope of the regression line is alpha (α) and is used to calculate the shear modulus as a function of loading frequency by:

$$G(f_2) = \frac{1 + \alpha f_2}{1 + \alpha f_1} G(f_1) \qquad 2.2$$

Where $G(f_2)$ is the predicted shear modulus at f_2, and $G(f_1)$ is the measured shear modulus at f1.

Undrained tests on mixtures of Ariake clay and silica-sand were performed by Yamada et al. (2008) using a cyclic torsional simple shear apparatus. Two different mixes were used in the investigation of the change in G/G_o with frequency. They found that at a strain level above 0.01%, the point when the behaviour is tending toward nonlinear, frequency has an effect on the normalized shear modulus. Over one log cycle between 0.1 and 1%, there is an increase of about 21% and 13% for pure Ariake clay and 30% Ariake clay-silica sand mixture at 1% cyclic strain amplitude respectively, and an increase of 6% and 9% at 0.1% strain amplitude for the same soils. Figure 2.7 also shows the other strain amplitudes at which the pure Ariake clay ‘C’ (ACC100) and 30% Ariake ‘C’ clay-sand mix (ACC30) were tested.

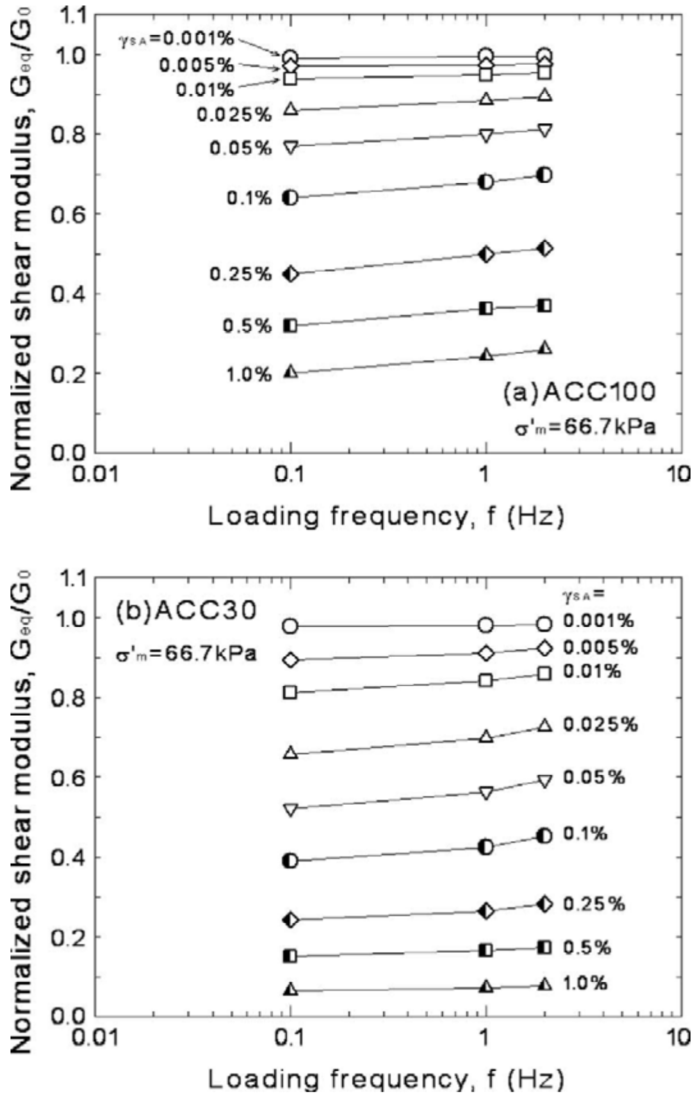


Figure 2.7: Change in normalized shear modulus with loading frequency for various cyclic strain amplitudes for a) ACC 100 and b) ACC30 (after: Yamada et al., 2008).

A sensitive clay from eastern Canada, St. Alban clay, was tested in a Hardin-type resonant column apparatus under isotropic and anisotropic conditions (Lefebvre et al., 1988). Samples were taken at 4.0 m and 7.9 m. The specimens were reconsolidated at the in-situ vertical stress and the in-situ vertical stress plus a simulated embankment load of 55 kPa. With the addition of the embankment load, the specimen from 4 m became normally consolidated while the specimen from 7.9m reaches the 1-dimentional vertical yield stress. In the specimens that were normally consolidated, consolidation just beyond the vertical yield stress actually increases the small strain shear modulus due to changes in void ratio and effective stress.

2.4 Classification of Sensitive Clay

Though it is agreed that sensitivity is the ratio of undisturbed to remoulded strength at the same water content, the definition of what constitutes a low, medium or highly sensitive clay varies. Skempton et al. (1952) showed that most clays, excluding heavily overconsolidated clays, exhibit some sensitivity and produced a scale based on their findings, which is shown in Table 2.2.

Table 2.2: Classification of sensitive clays from Skempton et al. (1952).

S_t	Definition
1	Insensitive
1-2	Low sensitivity
2-4	Medium sensitivity
4-8	Sensitive
> 8	Extra sensitive
> 16	Quick clays

Most early work on sensitive clays was conducted in Norway, and Norwegian clays exhibited very high sensitivities. The scale was later amended by Rosenqvist (1953) to better fit the sensitivity range of Norwegian clays, which is provided in Table 2.3.

Table 2.3: Classification of sensitive clay from Rosenqvist (1953).

S_t	Definition
1	Insensitive
1-2	Slightly sensitive
2-4	Medium sensitive
4-8	Very sensitive
8-16	Slightly quick
16-32	Medium quick
32-64	Very quick
> 64	Extra quick

The USA scale as reported by Hotlz et al. (1981) follows Skempton fairly closely and is reproduced in Table 2.4.

Table 2.4: Classification of sensitive clay from Holtz et al. (1981).

S_t	Definition
2-4	Low sensitivity
4-8	Medium sensitivity
8-16	High sensitivity
>16	Quick

More recently, the Swedish Geotechnical Institute has published a scale based on data from local sensitive clays (Rankka et al., 2004), which includes fewer ranges of sensitivity as displayed in Table 2.5.

Table 2.5: Classification of sensitive clay from SGI (Rankka et al., 2004).

S_t	Definition
< 8	Low sensitivity
8-30	Medium sensitivity
> 30	High sensitivity

The Canadian practice is usually reflected in the guidance provided in the Canadian Foundation Engineering Manual (CFEM). The most recent version of the manual (CFEM, 2006) originally presented one sensitivity classification but later issued an errata, which are provided in Table 2.6 a) and b).

Table 2.6: a) Sensitivity Classifications in the CEFM (2006) b) Sensitivity Classifications in the CEFM (2006) Errata

a)

S_t	Definition
< 10	Low sensitivity
10-40	Medium sensitivity
> 40	High sensitivity

b)

S_t	Definition
< 2	Low sensitivity
2-4	Medium sensitivity
4-8	Extra (high) sensitivity
> 16	Quick

Most clays, especially normally consolidated clays, lose some of their strength when remoulded and exhibit sensitivities of 2 to 4, even 4 to 8 is frequent (Abuhajar et al., 2010).

2.5 Remoulding Methods

Tests that can be used to determine sensitivity in the lab include the mini lab vane, unconfined compression and the fall cone. When determining sensitivity using these test methods, it is necessary to perform a test on an undisturbed and a remoulded specimen.

ASTM specifies remoulding methods for the mini lab vane and the unconfined compression. For an unconfined compression test, remoulded specimens may be prepared from a failed undisturbed specimen. The failed specimen is wrapped in a thin rubber membrane and worked thoroughly without trapping air in the specimen, with a uniform density and the same void ratio and water content as the intact specimen and then placed in an appropriate mold. The mini lab vane standard suggests that the vane be rotated 10 to 15 times and then a residual strength (which is noted that it may be higher than the remoulded strength) is found less than one minute after rotation. For sensitive soils, two rotations may be enough and, if used, multiple tests with different numbers of rotations should be done to determine if less than 5 to 10 rotations are appropriate. The sample may also be remoulded by hand while wrapped in a thin rubber membrane to prevent a change in water content and then tested.

The 2501-110 CAN/BNQ fall cone standard for determining sensitivity states that the remoulded sample should be made from unused portions of the sample. The sample is kneaded in a bowl with a spatula until it is thoroughly remoulded (defined as when the soil has a constant consistency and is homogeneous) and then transferred to the test container in three separate layers to avoid air bubbles in the specimen.

Devenny (1975) investigated multiple different methods of remoulding and compared the resulting undrained shear strengths using the lab vane. A mechanical mixer was used to remould specimens at various mixing times. It was found that the undrained shear strength did not change for Leda clay after 20 min, and for Labrador clay, after only 15 min. Devenny proposed using 'apparent sensitivity' and defined it as:

Apparent Sensitivity, $S_{ta} = \frac{U}{A}$ **2.3**

Where A is the shear strength after 15 revolutions of a standard laboratory vane and U is the shear strength found after complete remoulding in the mechanical mixer.

Yong and Tang (1983) remoulded specimens of Champlain clay in undrained DSS using continuous stress reversals. After a number of cycles, the loading stress-strain curve became repetitive and similar after each continuing cycle. One advantage of this method of remoulding is that the energy input can be quantified. They pointed out that the "100 %" remoulded state that the specimens achieve is only applicable for the simple shear apparatus used in the study and that it may not be the same as a remoulded state reached by hand remoulding.

2.6 Conclusions

The cyclic behavior of sensitive clay is an important design consideration for the seismic design of foundations in Eastern Canada. There have been a limited number of studies on the cyclic behaviour of Eastern Canadian sensitive clays, especially using simple shear testing. The cyclic simple shear test is the test that most closely reproduces earthquake stress conditions (Kramer, 1996), and more accurately evaluates the soil behavior than the other most commonly used cyclic test, the cyclic triaxial. In addition, the method of remoulding and energy used for remoulding sensitive clay specimens are important for determining the clay sensitivity. Reviewing the previously published methods of remoulding, there is not one method that is both used outside of one testing apparatus so that the remoulded soil can be used in any test, and where the energy used to remould the soil is quantified.

A method of remoulding that is relatively quick and inexpensive but can also measure the energy used to remould the sample was developed as a part of this study. In addition, stress- and strain-controlled cyclic simple shear tests are performed on undisturbed as well as remoulded sensitive clay specimens. Strain-controlled tests are done at varying frequencies and strain amplitudes in order to determine the degradation behaviour for a variety of conditions. Stress-controlled tests are performed at 1 Hz and the results are compared with results from a sensitive clay from Quebec. Tests are performed at a yield stress ratio ($\sigma'_{vy}/\sigma'_{vo}$) of 1 and 2 in order to determine the behaviour of the sensitive clay at the in situ overburden pressure (YSR=2) and at the yield pressure (YSR=1).

Chapter 3

3 Experimental Setup

The experimental program involved testing sensitive clay specimens retrieved from the Ottawa area employing cyclic simple shear, fall cone tests (CAN/BNQ 2501-110/2006) and a resonant column test. A standardized method for remoulding the specimens was developed and used in preparing all the disturbed specimens.

Ten shelby tube samples were retrieved from one large construction site in Ottawa and were tested in laboratories at the Golder Associates Burnaby office and the University of Western Ontario using cyclic simple shear and resonant column equipment. Each shelby tube was extruded in 100 mm sections, wrapped in cellophane, cheesecloth and then waxed to prevent moisture loss and disturbance during transport. The dynamic and static strength of these soils are analyzed in this study. A table describing the parameters of each test is shown in Table 3.1. Only representative testing results are shown in this chapter. However, all other test results are collected in appendix A.

3.1 General Properties

A representative specimen of each shelby tube was used to find the general properties of that sample. The Liquid and Plastic limit were determined using the procedure described in ASTM D4318 (2010). After a sample was removed from the wax in preparation for test, a representative piece of the sample was weighed and dried in order to determine the natural water content. The grain size distribution was determined by hydrometer analysis according to ASTM D422 (2007) with percent clay sized particles at <0.005 mm. The unit weight of the clay was estimated by finding the volume and weight of a sample recently removed from its wax. The in situ vertical stress was also estimated using the calculated unit weight and estimate of the water table depth. The plasticity index, liquidity index, and percent clay sized particles fall within the averages stated in the literature review (Mitchell and Soga, 2005; Mitchell and Klugman, 1979). Leda clay contains rock flour and amorphous materials such as iron, alumina and silica. The specific gravity is high because of the certain percentages of each material. Two previous studies have found specific gravities of 2.75 (Mitchell and King, 1977) and 2.80 (Yong and Tang, 1983) for two different sample of Champlain Sea clay.

Table 3.1: General properties of the samples tested.

Sample ID	Depth (m)	Natural Water Content, w_c	Moist Unit Weight (kN/m^3)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	Liquidity Index, LI	Specific Gravity, G_s	Percent Clay-sized Particles	Estimated Overburden Pressure (kPa)
SHL 1	15.14 - 15.65	66.1	15.8	60	25	35	1.2	2.83	78	103
SHL 2	9.14 - 9.65	75.0	16.3	72	24	48	1.1	2.86	72	149
SHL 3	19.81 - 20.32	53.0	16.3	34	19	15	2.3	2.87	56	172
SHL 4	10.5 - 10.96	58.8	16.0	55	22	33	1.1	2.90	66	91
SHL 5	3.96 - 4.47	82.0	15.6	64	24	40	1.5	2.86	75	39
SHL 6	7.5 - 7.96	72.9	16.3	49	24	25	2	2.85	80	61
SHL 7	16 - 16.46	62.0	16.3	56	24	32	1.2	2.85	72	116
SHL 8	7.62 - 8.13	74.4	16.4	64	25	39	1.3	2.82	72	67
SHL 9	7.5 - 7.96	83.4	16.3	69	26	43	1.3	2.88	75	61
SHL 10	10.5 - 10.96	82.0	16.3	58	24	34	1.7	-	84	83
09-M5	6.55 - 5.25	76	15	73	24	49	1.2	-	80	39

A powder X-ray diffraction analysis was also performed on a specimen taken from SHL 3. The experimental parameters of the diffractometer are listed in chlorite ($Mg_{5.0}Al_{0.76}Cr_{0.23}Al_{0.96}Si_{3.04}O_{10}(OH)_8$), phlogopite ($KMg_3(Si_3Al)O_{10}(OH)_2$), and muscovite ($(K_{0.82}Na_{0.18})(Fe_{0.03}Al_{1.97})(AlSi_3)O_{10}(OH)_2$). Ethanol was added to the sample in order to mount it.

Table 3.2. Matching phases include quartz (SiO_2), anorthite ($(Ca,Na)(Si,Al)_4O_8$), sanidine ($K(Si_3Al)O_8$), magnesiohornblende ($Ca_2(Mg,Fe^{+2})_4Al(Si_7Al)O_{22}(OH,F)_2$), kaolinite ($Al_2Si_2O_5(OH)_4$), chlorite ($Mg_{5.0}Al_{0.76}Cr_{0.23}Al_{0.96}Si_{3.04}O_{10}(OH)_8$), phlogopite

($KMg_3(Si_3Al)O_{10}(OH)_2$), and muscovite ($(K_{0.82}Na_{0.18})(Fe_{0.03}Al_{1.97})(AlSi_3)O_{10}(OH)_2$). Ethanol was added to the sample in order to mount it.

Table 3.2: Experimental parameters for the diffractometer.

Type of radiation:	CoKα1 = 1.78897 Å
Power:	40 kV x 35 mA
Beam diameter:	Conventional
Type of scan:	Step Scan
Step size:	Effectively 0.04°/step
Step time:	5 seconds/step
2θ range:	5°-90°

The test was performed at the Department of Earth Sciences at the University of Western Ontario and the results are based on a best match for the pattern from the International Center for Diffraction Data (ICDD) database.

3.2 Oedometer Tests

Oedometer tests were performed to determine the gross yield stress, the ICL, the in-situ yield stress ratio, and the breakdown of structure after remoulding. Fourteen tests were performed and are listed with each test's parameters in Table 3.3. Samples SHL 2 to SHL 5 were performed at UWO in 50 mm diameter and 13 mm high metal rings, all other oedometer tests were performed at Golder Associates' Laboratory using 63 mm diameter by 25 mm high rings. The load increment ratio was 1 for all tests except the ICL for which it was 0.5. The loading schedule followed in each Oedometer test is shown in Table 3.4 a) and b). The ICL and remoulded tests were performed at the natural water content of the specimen to keep the pore water chemistry the same as the intact specimens. All remoulded tests had liquidity indices greater than 1. A light layer of vacuum grease was applied to the oedometer ring before testing. Drainage was permitted through the top and bottom of the specimen. The yield stress was found using the Casagrande and confirmed by the strain energy method (Becker et al., 1987).

The yield point of most specimens occurred at 2-3% strain or less, meaning that most specimens were of fair quality (Sample Quality Designation from Terzaghi et al. 1996). SHL 2 and SHL 3 were performed on specimens at the very bottom of the shelby tube. The large strains before yield of SHL 2 and SHL 3 are most likely because these specimens were taken from a locally disturbed portion of the tube. For the type of sample extraction (3" Shelby tube) and distance of transport, the samples are considered to have been in good condition.

Three remoulded tests were performed at their natural water content as it was higher than the liquid limit. Burland (1990) suggests remoulding at approximately 1.0 - 1.5 times the liquid limit (LL) without air or oven drying, and within the natural pore-water of the soil. The ICL was remoulded at 1.58*LL, and the remoulded specimens of SHL 3, SHL 5 and M5 were remoulded at 1.6, 1.3, and 1.2 times the LL respectively. The ICL is also used within the sensitivity framework in order to compare the structure present in remoulded and intact specimens. Within the sensitivity framework, the standard used in Cotecchia and Chandler (2000) is Burland's (1990) published relationship between vertical stress and I_v. Using Burland's ICL equation, which is independent of water content, within the sensitivity framework as applied to the clay in this study results in a good comparison with field vane data.

Table 3.3: Oedometer tests performed. (Re) denotes a test on remoulded clay.

Sample ID	Depth (m)	Initial Water Content, w_c	Estimated Overburden Pressure (kPa)	Yield Pressure (kPa)	Insitu Yield Ratio
SHL 1	15.55 - 15.50	66.1	103.2	160	1.6
SHL 2	9.6 - 9.5	67.8	149.4	170	1.1
SHL 3	20.27 - 20.17	54.0	172.5	180	1.0
SHL 3 (Re)	19.97 - 19.87	57.1	"	-	-
SHL 4	10.81 - 10.71	60.1	90.7	120	1.3
SHL 5	4.42 - 4.32	85.6	39.2	125	3.2
SHL 5 (Re)	4.32 - 4.22	84.0	"	-	-
SHL 6	7.96 - 7.91	85.2	61.1	105	1.7
6 (ICL)	7.61 - 7.56	77.0	"	-	-
SHL 7	16.46 - 16.41	65.8	116.2	200	1.7
SHL 8	7.62 - 7.98	73.7	67.4	130	1.9
SHL 9	7.86 - 7.81	91.9	61.1	100	1.6
SHL 10	10.5 - 10.96	89.9	82.5	100	1.2
M5 (Re)	5.52 - 5.41	84.04	39.0	-	-

Table 3.4: a) ICL and b) ASTM Load Increments.

a)

Step	Vertical Stress (Psf)	Vertical Stress (kPa)
1	60.0	3.0
2	90.0	4.5
3	135.0	6.8
4	202.5	10.1
5	303.8	15.2
6	455.6	22.8
7	683.4	34.2
8	1025.2	51.3
9	1537.7	76.9
10	2306.6	115.3
11	3459.9	173.0
12	5189.9	259.5
13	7784.8	389.2
14	11677.2	583.9
15	17515.8	875.8
16	26273.6	1313.7

b)

Step	Vertical Stress (Psf)	Vertical Stress (kPa)
1	100	5
2	250	12.5
3	500	25
4	1000	50
5	2000	100
6	4000	200
7	8000	400
8	16000	800
9	32000	1600
10	16000	800
11	4000	200
12	1000	50
13	250	12.5

3.3 Remoulding

Remoulded specimens were prepared from 100 mm samples cut from the length of a shelby tube. The specimens were prepared by leveling the ends and using the trimmings to determine the water content then the specimen's weight and dimensions were recorded. The apparatus was set up in a Plexiglas and steel box with a 25 ton hydraulic ram. The hydraulic ram has a tolerance of +/- 5 kPa. The ram was powered by a variable rate motor. Before pushing the specimen through the remoulding apparatus, a test was done to determine the rate at which the ram moves. A small, cantilevered piece of metal was clamped on to the ram and a LVDT was set upon the piece of metal. As the ram moved down, the time and position of the ram were recorded and then the rate was calculated. The same rate was then used to push the soil through the remoulding apparatus. The rate had to be measured for each specimen due to the variable-speed motor and its use at other speeds for different experiments in between remoulding tests. Tests were performed at a rate of approximately 7.1 mm/min (+/- a maximum of 0.4 mm/min).

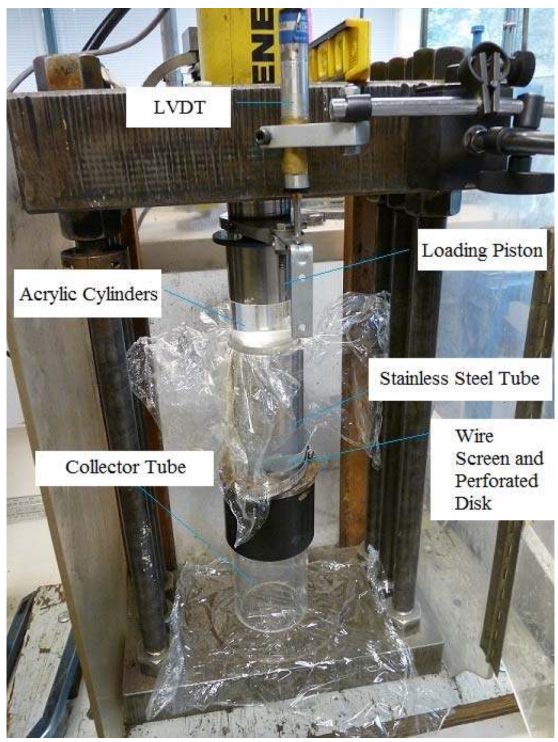


Figure 3.1: Remoulding Apparatus

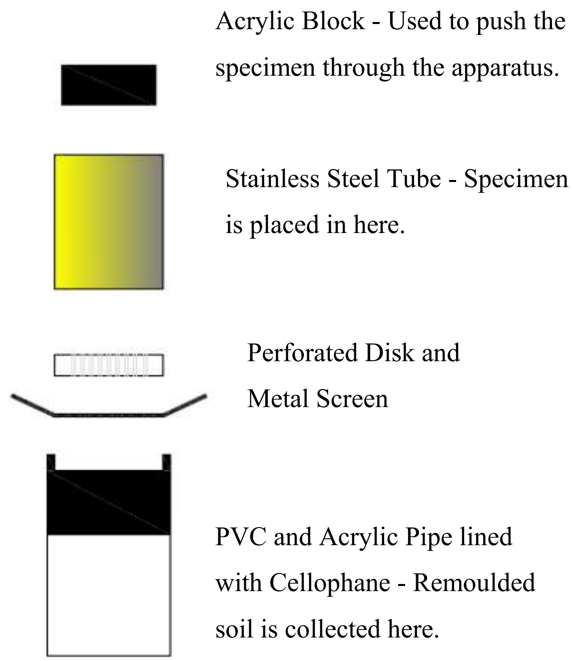


Figure 3.2: Components of the remoulding apparatus.

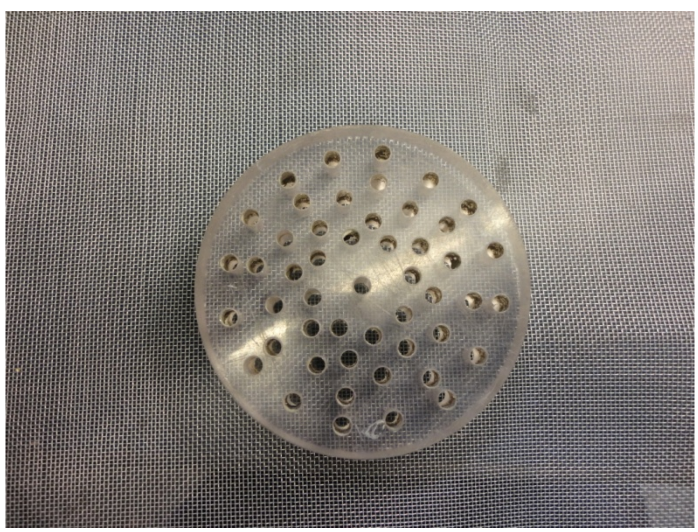


Figure 3.3: Perforated plastic disk and wire mesh.

The acrylic disk was 76 mm in diameter, 12 mm in height, and a 5/32” drill bit was used to bore holes in a starburst pattern. The starburst pattern was laid out with the dimensions as shown in Figure 3.4. The centre of the outer hole to the edge of the disk measured 0.7 mm. The woven, steel, wire mesh had a wire diameter of approximately 0.25 mm and a gap size (wire centre-to-centre) of 1 mm. In one direction the wire mesh had 28 holes per inch and in the other it had 26 holes per inch. The disk was placed so that before extrusion the sample rested on it, and the wire mesh was placed under the acrylic disk. The disk and wire mesh were held in place on the end of the steel tube by a stainless steel hose clamp. A 3” shelby tube extruder fitting rested on top of the collector tube and the wire mesh and disk were inserted into the indentation of the fitting as shown in Figure 3.5. Remoulding a sample using a disk with a smaller size of holes was attempted, but instead of extruding though the disk and wire mesh, the clay extruded up past the blocks and ram used to push the sample. Extrusion of the clay in the opposite direction intended was not an issue with the hole size that was used. In another test an additional wire mesh rotated at 45 degrees was used to test its affect on the remoulded strength but it was found that the remoulded strength did not change.

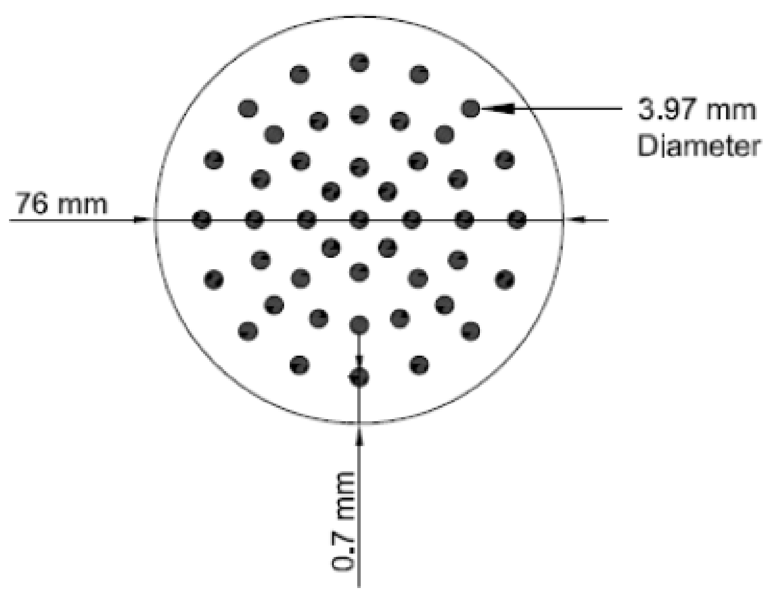


Figure 3.4: Starburst pattern of holes drilled into the acrylic disk.

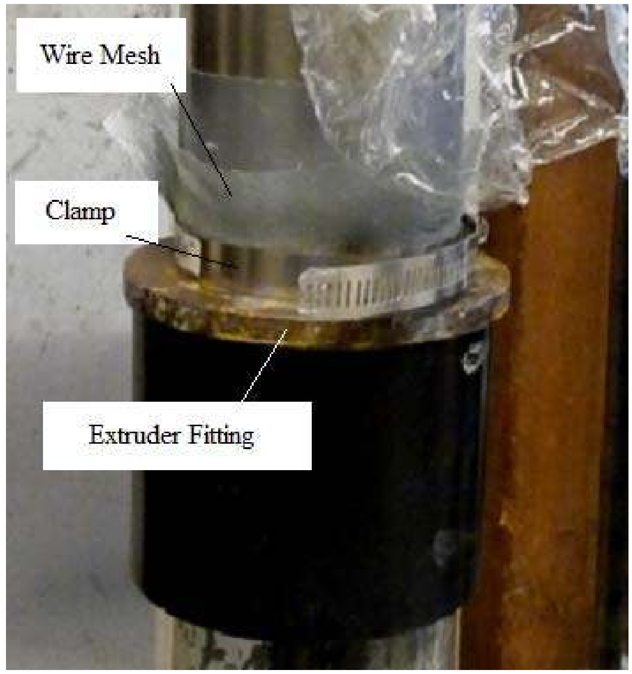


Figure 3.5: Close up of the connection between the steel tube, disk, wire mesh, and extruder fitting.

Before performing a test, the specimen was placed in the 100 mm length of stainless steel shelby tube, the acrylic blocks were weighed and then stacked on top of the specimen to meet the loading piston. When the data was processed, the weight of the acrylic blocks was also subtracted from the load (in Newtons) on the sample. A computerized data acquisition system was used to record the pressure (in kPa) applied by the ram versus time. The acquisition system was initiated and then the motor, the specimen was pushed by the ram through the perforated disk and the metal screen, and then collected in cellophane within the PVC pipe.

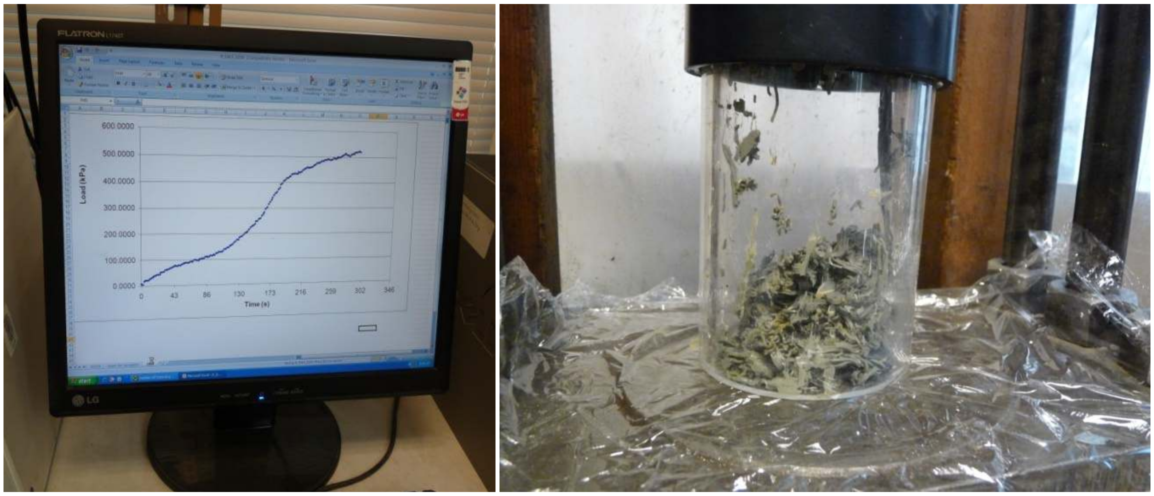


Figure 3.6: Output of computerized data acquisition system and the clay after being extruded through the remoulding apparatus.

The cellophane was wrapped around the remoulded soil to prevent moisture loss after the remoulding process was complete. Before the remoulded soil was used in subsequent testing, a specimen was collected to determine the water content. For direct cyclic simple shear (DCSS), fall cone, and oedometer tests the clay was placed in thin successive layers with a spatula in order to avoid air bubbles within the specimen. A conventional spatula was found to not be at an angle that could accommodate the shape of the DCSS specimen, cylindrical oedometer rings or the fall cone cup and produce evenly layered specimens. So a spatula made up of a plastic rod attached to a flat, metal disk with a diameter of 25 mm was used (Figure 3.7).

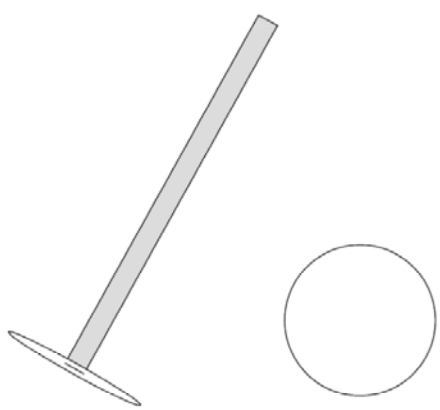


Figure 3.7: Modified spatula.

To process the data from the computer acquisition system, both the rate and remoulding data were collected into one spreadsheet. The rate was determined be finding the slope of the distance versus time plot. The rate is then multiplied by remoulding time and the displacement of the ram and height of the specimen is found. As seen in Figure 3.8, the pressure (kPa) - time (seconds) graph shows an initial gently increasing slope followed by a steep increase, a leveling off of the load and then a sharp drop once the motor is turned off and the ram is retracted. Since the specimen is not exactly the same size as the inside of the steel tube, a gentle increase in slope as shown in Figure 3.8 indicates the 'squishing' of the specimen to fill the annulus between it and the steel tube. The following increase in load represented by a steep increase in slope precedes the remoulding of the clay through the perforated disk and the wire screen represented by an approximately constant load.

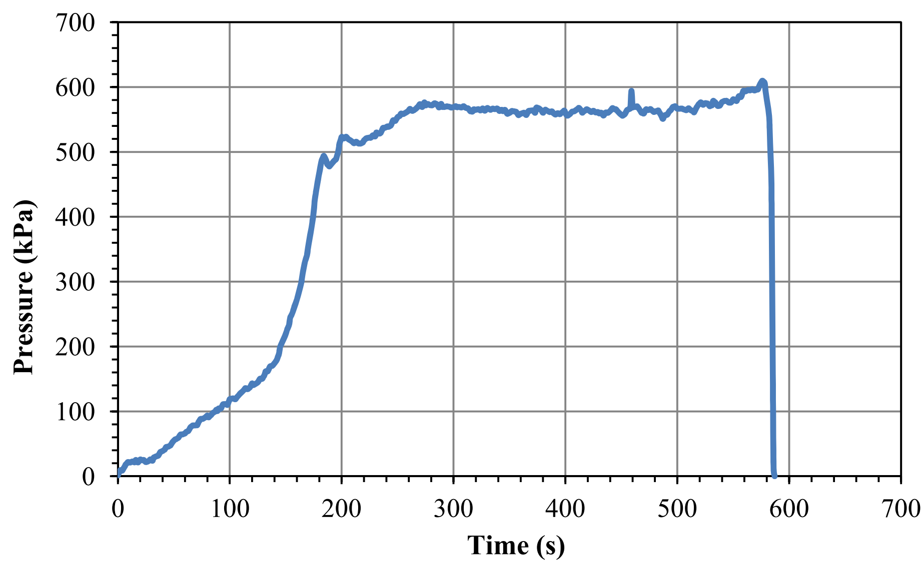


Figure 3.8: Pressure (kPa) versus time (s). Output graph of a remoulded test.

The pressure can be converted into force (N) by multiplying by the area of the ram and the distance travelled can be found, as mentioned previously, by multiplying time (min) with rate (mm/min). The area under a plot of force (N) versus distance (m) is equal to the cumulative work needed to remould the specimen (J). Friction between the wall and the specimen, the wall and the acrylic block, the specimen and the holes in the perforated disk and the wires of the screen are included in any unaltered force data. Friction between the wall of the tube and the specimen and between the wall and the acrylic block are the most significant sources of friction during remoulding. Data was collected from a specimen, after the ram had completed the process of 'squishing' the specimen to close the annulus between the tube and the specimen, with an open ended steel tube. This test was done to determine the friction between the wall and the specimen and the acrylic block and the tube wall.

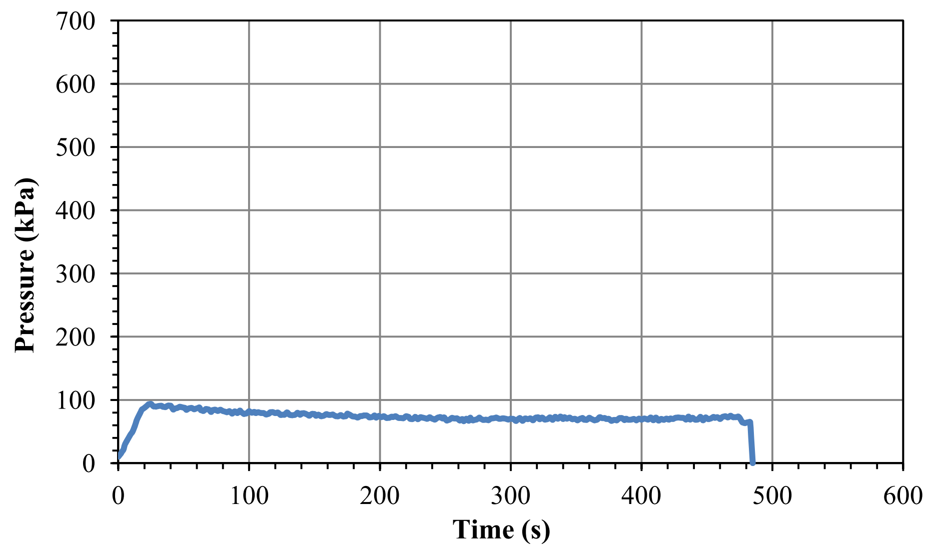


Figure 3.9: Output graph of the test to determine an estimate for friction between the wall and specimen, and the wall and acrylic blocks.

Figure 3.9 shows the test to determine wall-specimen, wall-block friction. The friction is highest at the beginning of extrusion and then decreases as the specimen is pushed out of the steel tube. The cumulative remoulding energy for the friction test was 12 J. Since the amount of energy needed is depends on the length of specimen pushed, this value is representative of a sample of approximately 60 mm. The samples that were remoulded had a length when flush with the tube wall ranging from 65 to 90 mm and the length of extruded specimen when flush with the tube wall ranged from 33 to 53 mm. Since the length of the initial specimens were greater than the friction test but the length extruded was less, the friction test was considered to be representative of the remoulded specimens so 12 J was subtracted from the total energy needed to remould each specimen.

The sensitivity of a clay is defined as the ratio of the intact shear strength to the remoulded shear strength at the same water content. Therefore, the water content for different samples was determined using the specimen trimmings and after remoulding and the results are presented in Figure 3.10. It can be noted from Figure 3.10 that the water content after remoulding was slightly higher than the water content determined from sample trimmings. It should be noted that the trimmings were taken from the end of the waxed sample and may have water content that could be slightly lower than the water content of the inner part of the sample. The outlier was the specimen from SHL 5, which was retrieved from an elevation above the water table and at the boundary between the brown, weathered crust and a grey, silty clay layer. It is likely the difference in trimmed and remoulded water contents was due to the softer, wetter clay being pushed out and remoulded while the hard, brown, weathered clay was not remoulded.

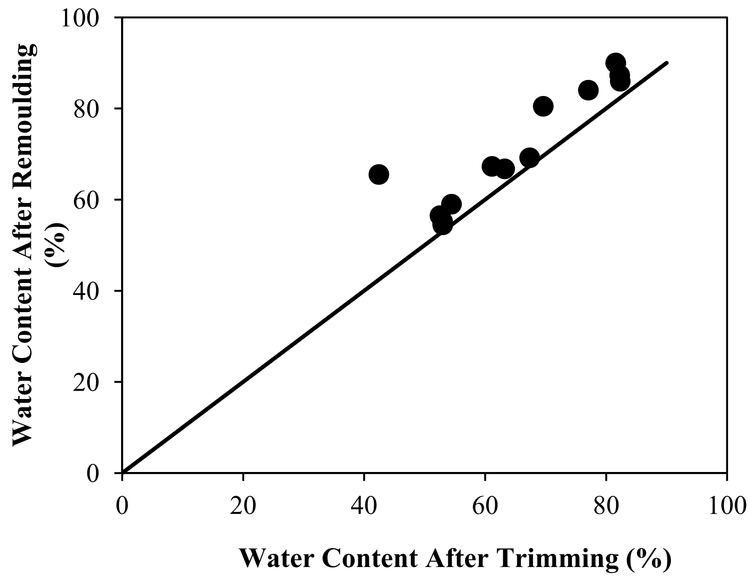


Figure 3.10: Water content of specimens before and after remoulding.

Though there is some variability in the energy to remould a specimen due to friction, and the length of specimen extruded, if the same set-up is used consistently the variability can be managed. Figure 3.10 shows that soft, homogeneous clay is more likely to have the same water content after and before being pushed through the remoulding apparatus. In general, the method described in this section is a useful way to measure the energy needed to remould a sensitive clay sample and to remould it in a way that is consistent between samples.

3.4 Simple Shear Tests

3.4.1 Sample Preparation

Intact samples were extruded from the thin tube sampler and cut into 10 cm high cylinders that were wrapped in plastic wrap, cheesecloth and then waxed for transport to the lab. Once the sample was needed for testing, the wax, cheesecloth and plastic were carefully removed and the sample trimmed to fit the direct cyclic simple shear platens. A stainless steel ring machined to the 70 mm diameter of the DCSS platens and 23.7 mm high was used to trim the sample to the proper dimensions. The specimen was then taken from the stainless steel ring and wrapped in plastic to prevent any loss of moisture. Trimmings were also collected to determine the water content.

3.4.2 Description of Apparatus Used

A GDS Electromechanical Dynamic Cyclic Simple Shear (EMDCSS) machine was used to perform all simple shear tests. The machine has a load capacity of 5 kN, with an accuracy of +/- 0.1% of the load cell range for both axial and shear and a 16 bit resolution (i.e. less than 0.4N). There are two linear variable differential transducers (LVDTs); one is specifically used to measure very small displacements. The axial displacement range is +/- 25mm and the lateral displacement (shear) is +/- 15mm with an accuracy of 0.1% FSO. The displacements measured with the low range LVDTs have a range of +/- 2.5mm for both axial and shear with an accuracy of less than 0.1% FSO (full scale output). The machine also complies with ASTM D6528 in accordance with NGI testing procedures.

The machine has two actuators: one is situated above the sample, which is fixed in the horizontal direction but free to move vertically; and one is situated below the sample, which is fixed vertically but free to move horizontally. Constant volume (simple shear) conditions are enforced by controlling the height of the specimen using feedback from the local axial transducer. Ridged boundary platens (Cambridge-type), wire-reinforced membrane (NGI-type) or stacked rings (SGI-type) can be used for lateral confinement. The SGI type stack of stainless steel, Teflon coated rings were chosen for this experimental program. Though the wire reinforced membrane seems to be the most common method used in practice, both the wire membrane and the stacked rings have been found to produce comparable results (Baxter et al., 2010).

Static tests as well as cyclic tests can be performed with the EMDCSS. Static tests have no specific shear-rate lower limit. Displacements can be up to 7.5 mm on either side, which translates to approximately 30% shear strain considering a specimen height of about 23 mm. Frequencies of up to 10 Hz can be achieved during cyclic loading. Measurement noise can be significant at high frequencies and at small amplitudes, and may influence the recorded data. The test results can also be affected by an initial static bias. Table 3.5 shows the variation of displacement amplitude versus frequency and force datum with and without an initial bias. Sine, square, triangular, haversine and other custom waveforms can be used.

Table 3.5: Limits to the DCSS machine.

Frequency (Hz)	with 5kN force datum		with zero kN force datum	
	Amplitude (mm)	Double Amplitude (mm)	Amplitude (mm)	Double Amplitude (mm)
0.1	50	100	50	100
0.2	50	100	50	100
0.5	26.5	53	26.5	53
1	13.3	26.6	13.3	26.6
2	6	12	6	12
3	2.8	5.6	4.4	8.8
4	1.6	3.2	3.2	6.4
5	1	2	2	4
7	0.5	1	1	2
10	0.25	0.5	0.5	1

The accuracy of biased tests varies from unbiased tests depending on the frequency used, as shown in Table 3.5. No biased tests were carried out in this experimental program.

3.4.3 Machine Set Up

The simple shear specimen apparatus is equipped with two stainless steel end platens and two porous stones. Additionally, a thin rubber membrane, 27 Teflon coated aluminum-brass rings, rubber o-rings, filter paper, vacuum grease and a vacuum mold were used during set up. Each porous stone had 2 mm pins inserted in them as shown in Figure 3.11.

Figure 3.11: End platens of the DCSS.

The porous stones were first cleaned by running water through them and then attached to the end platens with four screws. Filter paper cut to the size of the platens and with holes for the pins was placed on the porous stones. A thin layer of vacuum grease was spread on the side of the bottom platen. The rubber membrane was stretched over the platen and an o-ring was placed over the bottom of the platen and membrane. A metal ring was placed over the o-ring at the bottom of the platen to hold it in place. An additional o-ring was stretched around the metal ring and lightly covered in vacuum grease in preparation for the vacuum mold. The set-up with the platens, filter paper, metal ring, membrane and Teflon coated rings are shown in Figure 3.12.

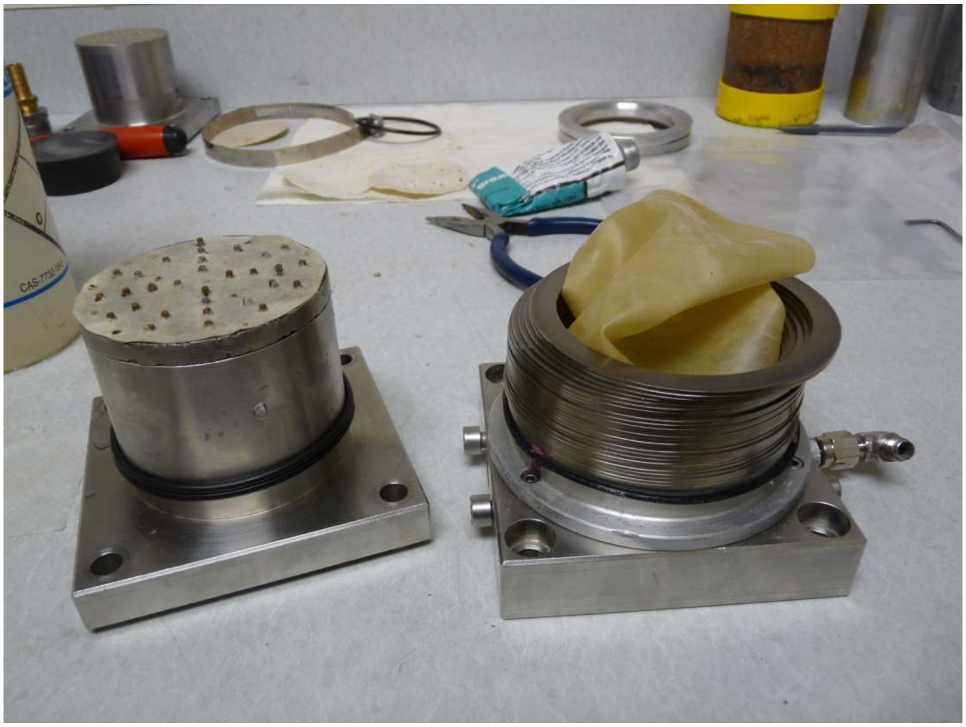


Figure 3.12: End platens with filter paper, metal ring, membrane and Teflon coated rings.

Three rings were used to hold the membrane in place over the platen and the remaining rings are stacked on top of them. The vacuum mold was then placed around the rings and with the o-ring sitting in a groove and held together by a clamp. With the rubber membrane stretched over the mold, a vacuum line was attached to it and the membrane was sucked close against the inside of the rings. Intact specimens were prepared as previously mentioned and then placed in the membrane and remoulded specimens were carefully placed in the membrane to a specified height. The top platen was lined up and then put on top of the specimen and the apparatus was secured in the simple shear machine. Figure 3.13 and Figure 3.14 show the last two steps.

Figure 3.13: Set-up with specimen inside mold.

Figure 3.14: Set-up in the DCSS machine.

3.4.4 Monotonic DSS Tests

Undisturbed, monotonic direct simple shear tests were performed on one specimen from SHL 1 to SHL 5 at the in-situ overburden pressure. Specimens SHL 6, 7 and 9 were tested at 100 kPa. The properties of the undisturbed soil specimens determined from the monotonic tests are listed in Table 3.6. One test was conducted on a remoulded soil specimen in the DCSS to determine its sensitivity but it was too difficult to keep the moisture content constant during consolidation and shearing so no more tests were performed for this purpose. The maximum S_u was determined from the peak strength and the residual S_u was determined after the strength had leveled off past peak. Failure at peak strength occurred within 2-4% strain which is similar to the strain at failure of other monotonic simple shear tests carried out on Champlain Sea clays (Lefebvre and Pfendler, 1996; Silvestri et al., 1989; Yong and Tang, 1983). Past peak horizontal stress the influence of non-uniform stresses on the response of the DCSS increases so strength values at large shear strains may not necessarily represent 'true' soil behaviour (DeGroot et al., 1994).

Table 3.6: Soil strength parameters determined from monotonic tests.

Sample ID	Max s_u (kPa)	Residual s_u (kPa)	Vertical Stress	Max s_u/σ'_{vc}
1-M-U	27	20	104	0.26
1-M-R	27.8	27.8	104	0.27
2-M-U	41	22	149	0.28
3-M-U	56	24	175	0.32
4-M-U	23	17	90	0.26
5-M-U	21	17	40	0.53
6-M-U	25	16	100	0.25
7-M-U	33	30	100	0.33
9-M-U	27	18	100	0.27

3.4.5 Direct Cyclic Simple Shear Tests

Strain-controlled tests were performed at the in situ overburden stress of each sample and stress-controlled tests were performed at 100 kPa. Strain-controlled tests were performed at varying frequencies and strain amplitudes in order to determine the effect of these varying parameters on the clays cyclic behaviour. A typical plot of cyclic stress ratio (CSR) versus number of cycles is shown in Figure 3.15 for a specimen from SHL 3 performed at 1 Hz and approximately 1% strain amplitude. All other strain-controlled tests had similar outputs but with varying strain amplitudes and rate of degradation with number of cycles. The upper limit of the DCSS machine was 10 Hz and the CSR vs. horizontal strain data show that there is a small stress bias that should not be present. These factors may have affected the stiffness data. Tests could not be run at 5% cyclic strain at 10 Hz due to the limitations of the DCSS machine.

Samples SHL 6 and 9 of the stress-controlled tests were performed with a vertical pressure equal to their yield pressure as determined by the oedometer which gives a yield stress ratio (YSR) equal to 1. Sample 7 was performed at a vertical pressure less than its yield pressure and had an YSR equal to 2. The cyclic stress ratio (τ_c / σ'_{vo}) of each stress-controlled test was varied between 0.1 and 0.34. In some tests the cyclic horizontal stress (τ_c) exceeds the monotonic soil strength. In these cases the cyclic strength is higher due to strain rate effects (Lefebvre and Pfendler, 1996). A typical plot of cyclic stress ratio versus number of cycles is shown in Figure 3.16 for a specimen from SHL 6 performed at 1 Hz and a CSR of 0.3. All other stress-controlled tests had similar outputs but with varying CSRs and amount of degradation with number of cycles.

Table 3.7: Strain-controlled cyclic simple shear tests

	Sample ID	Frequency (Hz)	Approx. Strain Amplitude	σ'_{vo}	Max τ_c/S_u
Strain-Controlled	1-C-U-A	1	0.1	104	0.35
	2-C-U-A	10	0.1	149	0.34
	2-C-U-B	10	1	149	1.35
	3-C-U-A	5	0.1	175	0.24
	3-C-U-B	5	1	175	1
	3-C-U-C	0.1	0.1	175	0.26
	3-C-U-D	1	1	175	0.98
	4-C-U-A	0.1	1	90	1.61
	4-C-U-B	0.1	5	90	1.6
	4-C-U-C	5	5	90	2.22

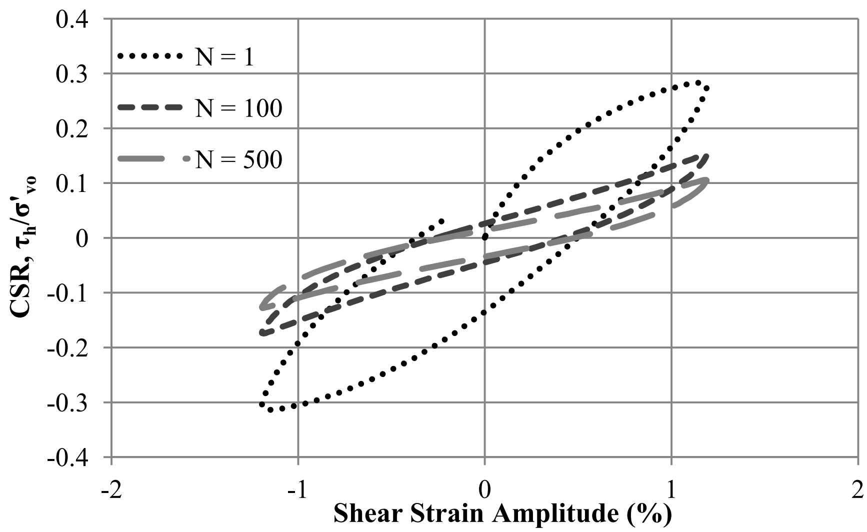


Figure 3.15: Strain-controlled test performed on SHL 3 at 1 Hz and approximately 1% strain amplitude.

Table 3.8: Stress-controlled cyclic simple shear tests

	Sample ID	Frequency (Hz)	σ'_{vo}	CSR	Max τ_c/S_u	τ_f (kPa)	N_f
						(at 3% strain)	
Stress-Controlled	6-C-U-A	1	100	0.1	Did not fail in 2000 cycles		
		1	100	0.2	0.81	20.0	169.3
		1	100	0.3	1.32	-32.7	10.8
	7-C-U-A	1	100	0.3	0.88	-29.1	34.8
		1	100	0.34	0.97	32.0	18.3
		1	100	0.25	0.72	23.7	75.3
	9-C-U-A	1	100	0.24	0.99	-26.1	69.8
		1	100	0.27	1.00	-26.6	66.8
		1	100	0.33	1.33	-35.3	6.8

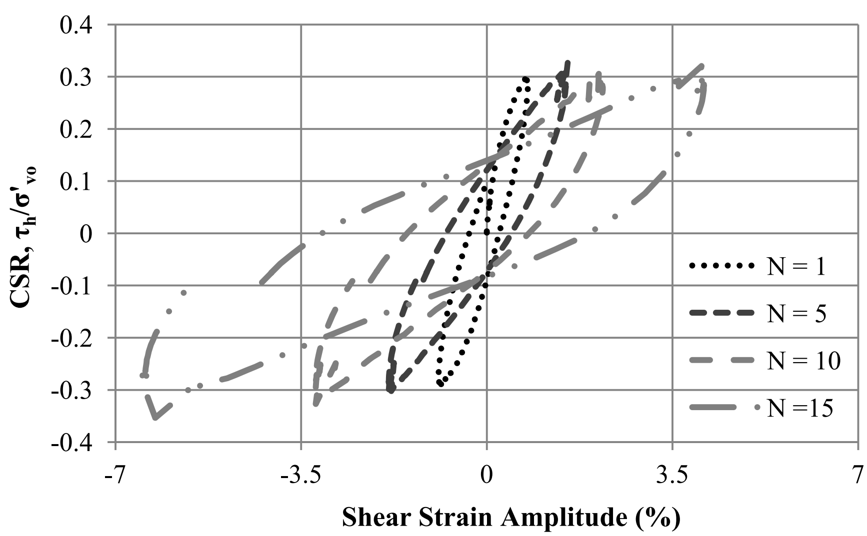


Figure 3.16: Stress-controlled test on SHL 6 at 1 Hz and a CSR of 0.3.

Resonant column and DCSS tests were performed on specimens from the same sample in order to evaluate their normalized stiffness degradation curves. The DCSS tests were performed at a vertical confining stress equal to the in-situ overburden stress at an average depth of 14.7 m, a unit weight of 16.6 kN/m^3, and a water table of 0.8 m below ground level. Table 3.9 shows the test parameters used. The specimen was consolidated for 2498 minutes before performing the test. The strain calculated from voltage at the 0.707 times the radius. Damping was calculated using the logarithmic decrement method. Because in a conventional resonant column test the strain is measured at the resonant frequency (33.5 Hz) and the DCSS test were performed at a much lower frequency (1 Hz) when both results are extrapolated to overlapping strains, the values of shear modulus do not agree. A method (Khan et al., 2010) to adjust shear modulus found from the resonant column to an equivalent shear modulus at 1 Hz was used to better match results from both tests.

Table 3.9: Resonant column and cyclic simple shear tests for characterization of the stiffness degradation curve.

Sample ID	Confining Stress (kPa)	Frequency (Hz)	Strain Amplitude (%)
P2-RC	72*	33	-
P2-U-0.1	106.4	1	0.1
P2-U-0.5	106.4	1	0.5
P2-U-1	106.4	1	1
P2-U-5	106.4	1	5

* the confining stress in the RC is a mean effective stress

3.4.6 Cyclic Behaviour of Remoulded Specimens

Four remoulded and four undisturbed specimens from the same borehole were used to investigate the effect of remoulding on cyclic behaviour. The remoulded specimens were pushed through the remoulding apparatus and then tested at the in situ vertical overburden stress at the same frequency (1 Hz) and varying strain amplitudes. The four undisturbed tests were conducted at the same vertical stress, frequency, and strain amplitudes as the remoulded tests. The results from each test were compared to understand the cyclic behavior of remoulded sensitive clay. Table 3.10 gives the properties of remoulded and undisturbed soil specimens. The water contents of the specimens were determined before and after the DCSS tests.

With increasing strain amplitude, all specimens showed a decrease in secant shear modulus. Even with consolidation of the remoulded specimens, the intact specimens had a higher initial stiffness (N=1) for each strain amplitude. The higher stiffness of the intact specimens is likely due to the intact structure of those samples while the remoulded specimens have been de-structured. The reduction in stiffness over 30 cycles is less for the remoulded specimens than for the intact specimens due to the difference in structure between specimens. There is a significant change between the initial and final water content of the remoulded specimens which results from consolidation of the remoulded specimens.

Table 3.10: Intact and remoulded-consolidated tests. All tests cycled though N=50.

	Sample ID	**Frequency (Hz)**	**Strain Amplitude**	**σ'_{vo}**	**w_c initial**	**w_c final**
Intact	M5-C-U-A	1	0.1	39	83.4	83.0
	M5-C-U-B	1	1	39	81.5	81.4
	M5-C-U-C	1	2	35	82.2	87.8
	M5-C-U-D	1	5	35	82.3	82.7
Remoulded	M5-C-R-A	1	0.1	39	90.0	73.1
	M5-C-R-B	1	1	39	80.5	68.6
	M5-C-R-C	1	2	35	87.4	72.0
	M5-C-R-D	1	5	35	86.0	68.0

Chapter 4

4 Results and Analysis

4.1 General Properties

All test samples had higher natural water content than liquid limit, which is characteristic of sensitive clays from eastern Canada. The specific gravity was fairly high and is possibly a reflection of the amount of rock flour (often mostly quartz and feldspar) in the sample. The grain size distributions of all samples were very similar. All samples had a negligible amount of soil retained on the #200 and 31 to 58 % passing a 0.001 mm size as determined by hydrometer testing. The percentage clay-sized particles was between 56 and 84% (clay-sized is considered less than 0.005 mm). Figure 4.1 presents the activity (plasticity index divided by the percent clay sized particles less than 2 μm in diameter) of test samples according to Skempton's (1950) activity definition. It can be noted from Figure 4.1 that most samples fall in the inactive category.

According to field vane tests on five of the samples, SHL 3 is the most sensitive and its activity falls well into the 'Inactive clays' region, while SHL 2 and 09-M5 are the most active. SHL 2 does have a low sensitivity (8) but the field vane estimate for 09-M5 is 23. The sensitivity of 09-M5 is an estimate based on a nearby borehole (BH-P6) and may not correctly represent the sensitivity of the clay from 09-M5. Eden and Kubota (1961) collected samples from the Ottawa area to compare different methods of measuring sensitivity. The activity values of the samples they collected are also plotted on the activity chart as shown in Figure 4.1. The dotted lines (Boring No. 1,2,3,4) represent average values of activity for the samples taken from each borehole by Eden and Kubota (1961) and the open circles represent the activity of each specimen. The authors commented that Leda clays typically have a higher plasticity index than Norwegian clays. Many of the specimens that the authors investigated were retrieved from the same general area around Ottawa, from which the specimens examined in this study were also retrieved. All the specimens in the present study compare well with the activity found for the samples tested by Eden and Kubota (1961).

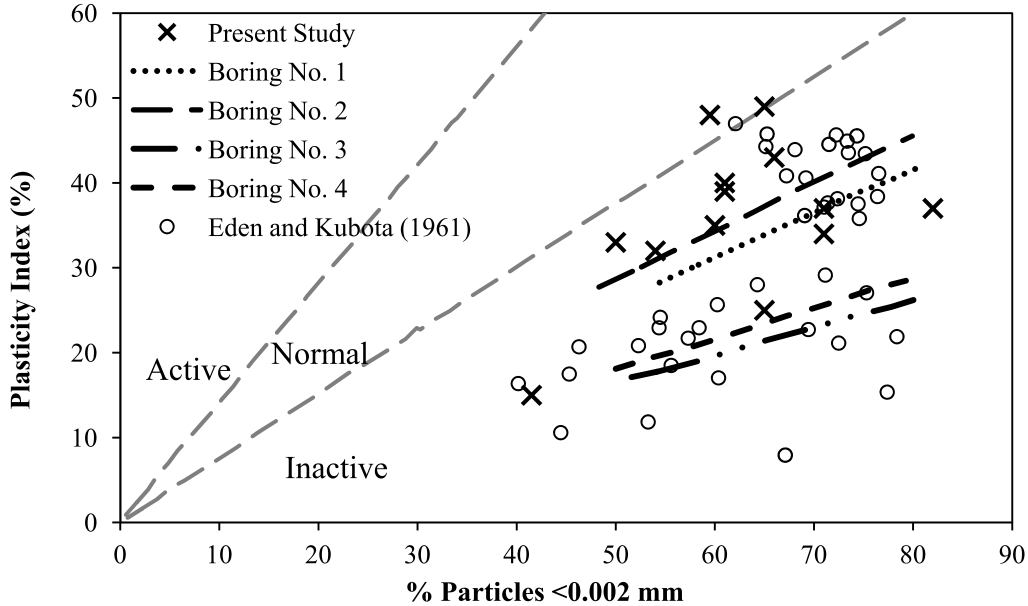


Figure 4.1: Activity of Leda clay in this study, and samples from four boreholes reported in Eden and Kubota (1961).

A powder X-ray diffraction analysis was performed on a specimen taken from SHL 3 and is shown in Figure 4.2. The qualitative results show that quartz, feldspars, hornblende, and micas were found present in the clay used in this study. Though chemical analysis would need to be done to confirm which minerals are present and in what quantities, these quantitative results contain similar minerals found in the study done by Locat et al. (1994) on various sensitive clays from Quebec including some from the Champlain Sea basin.

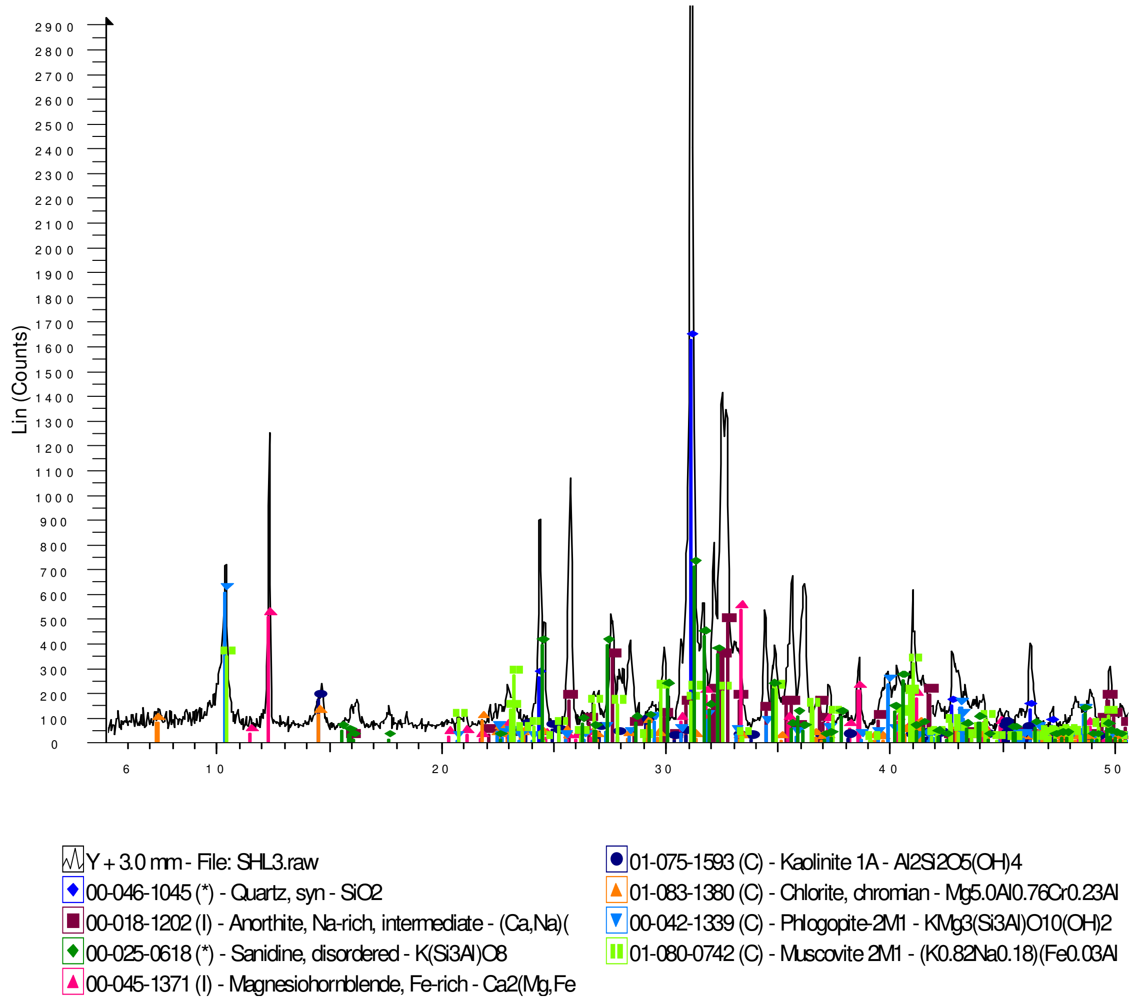


Figure 4.2: Results of the powder XRD analysis.

4.2 Results of Oedometer Tests

Figure 4.3 presents the conventional oedometer test results for 10 samples examined. The results of the oedometer tests show typical curves associated with eastern Canadian sensitive clays (Mitchell, 1970). The curves do not have constant compression indices (C_c) but rather C_c reduces with increasing stress past the yield stress. The change in slope immediately after the yield stress is well defined, similar to other eastern Canadian sensitive clays (Leroueil et al., 1983). Also, the slope is very steep resulting in initial high values of compressibility index (Hamilton and Crawford, 1959), which has been associated with the rupture of cementation bonds (Jarrett 1967; Walker and Raymond, 1968). Because of its structure, Leda clay has a low recompression index (Crawford, 1968). The recompression indices for all samples in this study varied from 0.011 to 0.072 for stresses between 5 and 50 kPa (lower than the yield stress). Specimens which were unloaded before the end of the test had swell indices varying from 0.028 to 0.12 over a range between the vertical stress at the end of compression and 50 kPa. The low swelling indices indicate the degree to which the structure of the clay has been disturbed. Most samples were under a pressure of 1500 kPa at the end of a test when unloading was performed while the in-situ pressures of the specimens ranged from 39 to 172 kPa so it is likely that the specimen has been significantly de-structured by the end of the test.

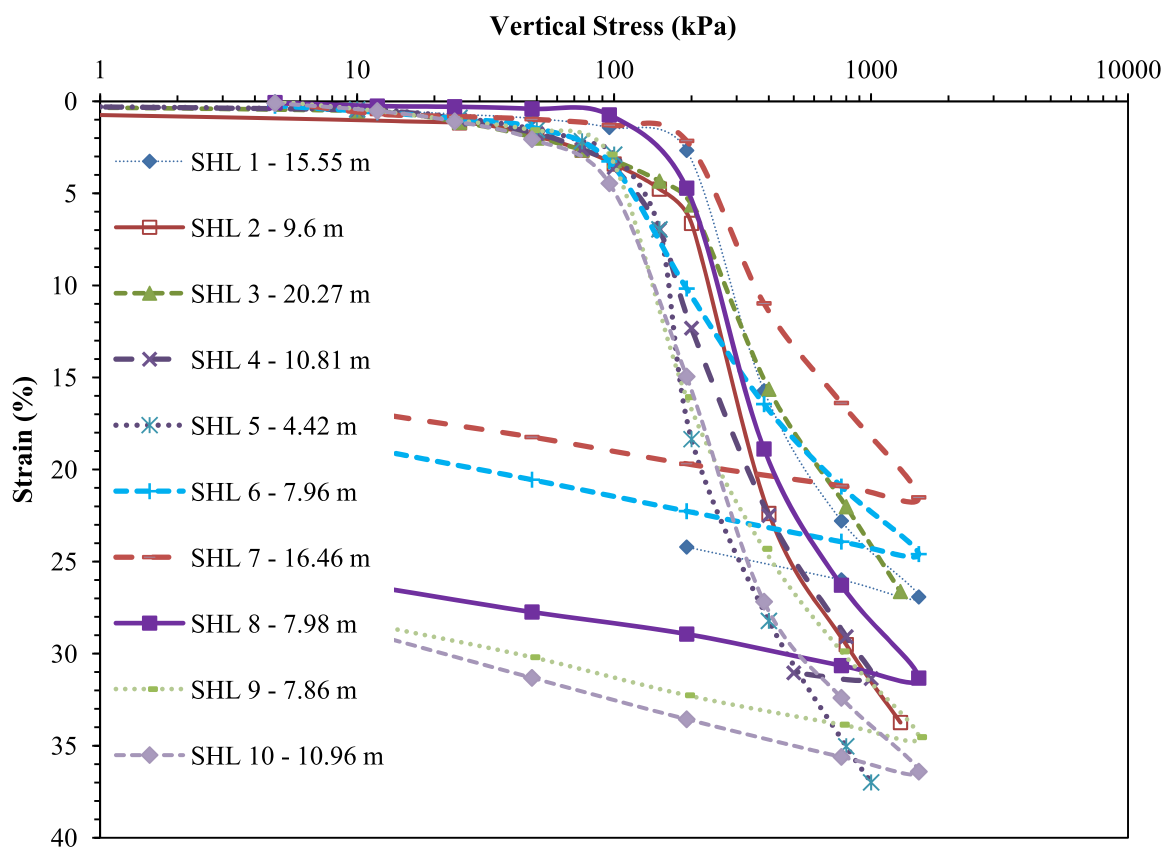


Figure 4.3: Conventional oedometer test results for all samples.

Figure 4.4 displays the normalized oedometer test results for the intact and remoulded tests using the void index according to the approach proposed by Burland (1990), which provides a relative measure of the structural breakdown of each specimen. In the same figure, the Burland's (1990) equation for predicting the ICL is plotted as 'Burland ICL' while the experimental ICL from SHL 6 is plotted as 'Exp ICL, 77%, 2' (77% is the natural water content and 2 is the liquidity index). The experimental ICL specimen was prepared from a sample from SHL 6 at its natural water content since its natural water content was 1.58 times its liquid limit. The experimental ICL matched the Burland's ICL starting at a vertical effective pressure of approximately 80 kPa which corresponds to Burland's (1990) observation that at pressures lower than approximately 100 kPa, compression curves tend to diverge but they converge at higher pressures.

The void ratio indices of the intact specimens were found using the experimental ICL values of void ratio at 100 and 1000 kPa. The sensitivity framework developed by Cotecchia and Chandler (2000) use the ICL and the yield point of post-sedimentation structure clays to determine the stress sensitivity. The stress sensitivity is a ratio of the yield stress of an intact sample to the stress on the ICL at the same void index. The stress sensitivity has been shown to be equal or very close to the value of strength sensitivity. In Figure 4.4, there is a cluster of specimens with stress sensitivities between 30 and 50, and around 10. For three specimens that also have corresponding field vane strength sensitivities, it is true that the stress sensitivity is very close as shown in Table 4.1.

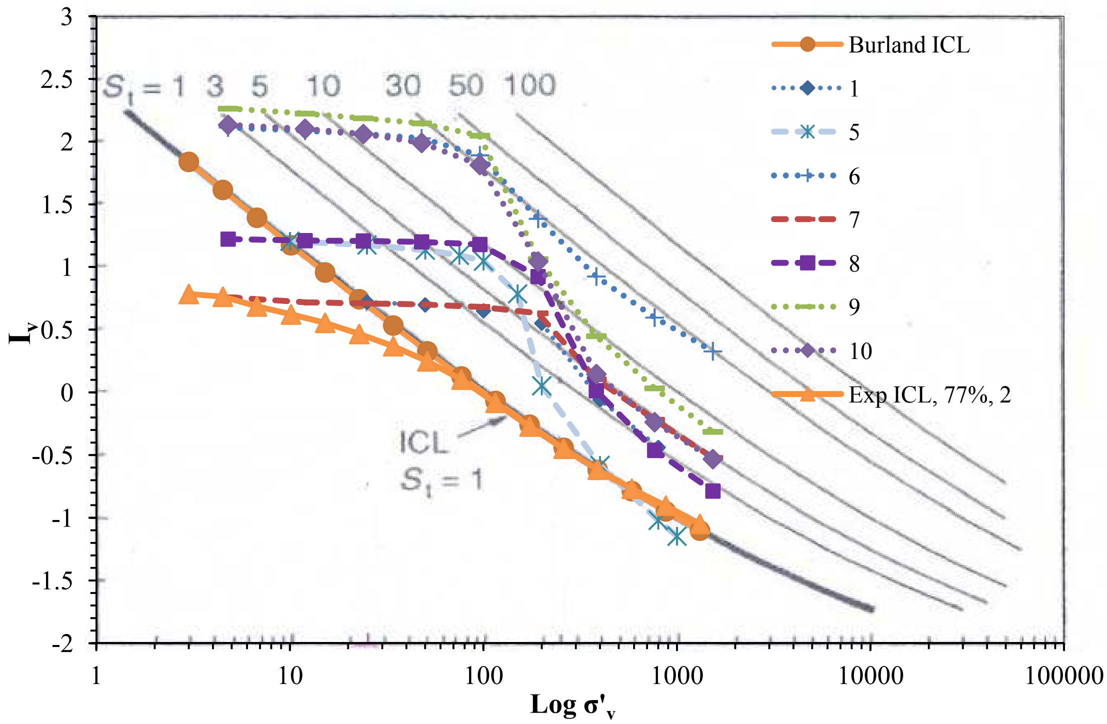


Figure 4.4: Oedometer results as applied to the sensitivity framework (background image taken from Cotecchia and Chandler (2000)).

Table 4.1: Comparison of field vane strength sensitivity, and stress sensitivity.

Sample ID	Depth (m)	Strength Sensitivity	Stress Sensitivity
SHL 1	15.14 - 15.65	6	6
SHL 5	3.96 - 4.47	8	8
SHL 8	7.62 - 8.13	7.3	11

Three remoulded specimens were subjected to conventional oedometer tests. Figure 4.5 shows the remoulded (using the remoulding apparatus) results compared with the respective intact curves of SHL 3 and SHL 5. SHL 3 and 5 were converted to void index using the intrinsic void ratio values from their respective remoulded tests. M5 and the Exp ICL were also plotted in I_v using the intrinsic values of void ratio found for each test.

The breakdown of structure by remoulding can be seen when comparing the remoulded and intact curves from SHL3 and SHL 5. Both intact curves collapse sharply to the remoulded curves once the soil is subject to stresses higher than the yield stress. As the structure is disturbed the intact curves converge with the remoulded curves indicating that the structure has been completely destroyed. The remoulded lines do not have a well defined yield point like the intact specimens as the structure the specimen once had has been removed by the process of remoulding.

Initial water content affects the position of the ICL (Cerato and Luteneger, 2004). This can be seen when the void ratio normalized by the void ratio at the liquid limit (e/e_L where $e_L=G_s*LL$) is plotted against log vertical pressure in Figure 4.6. As the liquidity index increases, the remoulded plot shifts higher. The e_L varies for each remoulded specimen because the liquid limit was found to be different for each. Since the remoulded tests have various initial water contents and liquid limits, it may be clearer to plot the data in e/e_L for a better visualization of the differences between samples as in Figure 4.7. However, it is shown in both Figure 4.6 and Figure 4.7 that the remoulding process has significantly destroyed the in-situ structure of the clay.

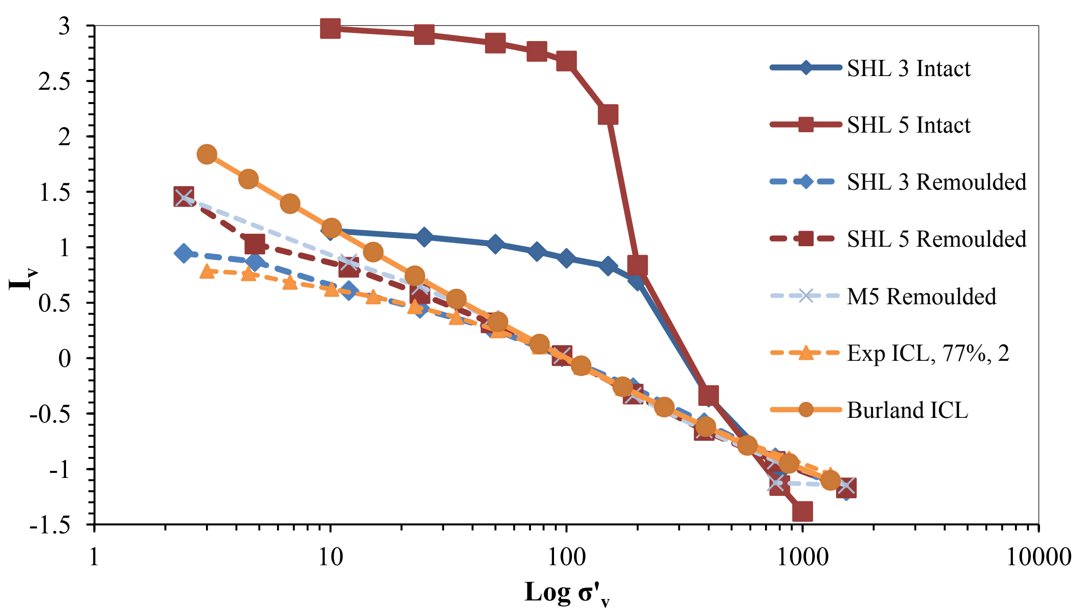


Figure 4.5: Breakdown of structure with reference to the ICL.

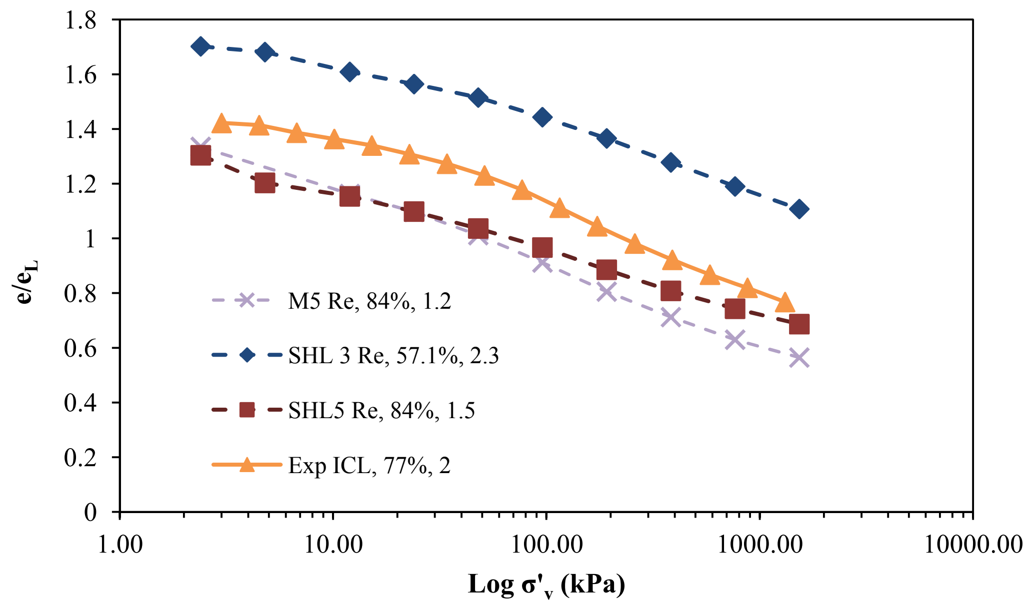


Figure 4.6: Difference in initial water content between remoulded specimens.

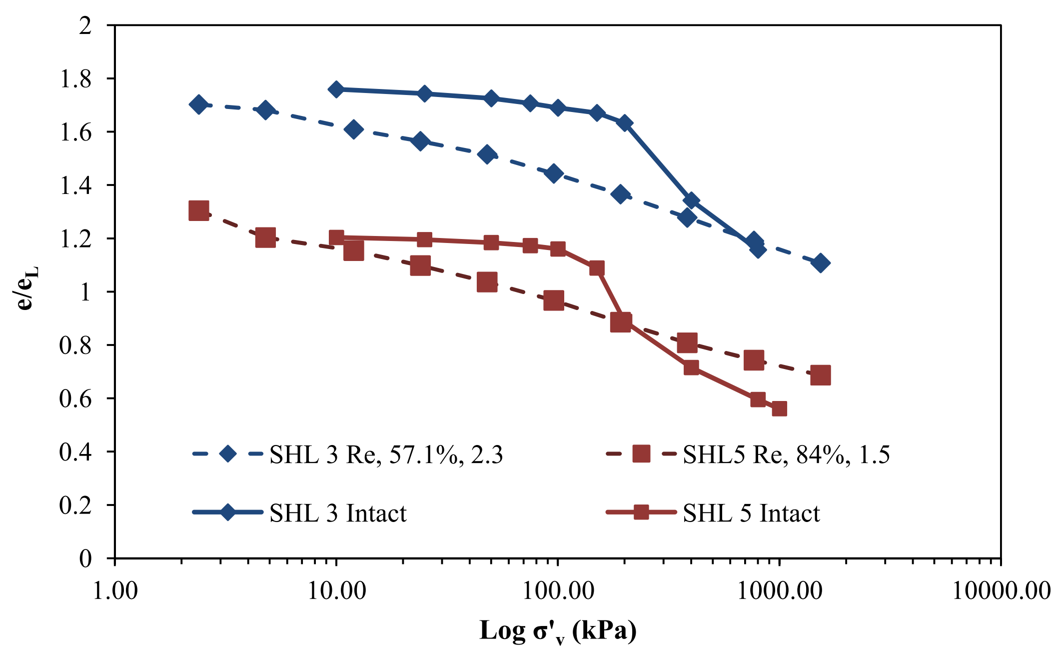


Figure 4.7: SHL 3 and SHL 5 intact and remoulded specimens.

4.3 Tests on Remoulded Clay

The energy used to remould each sample was calculated from the area under the distance (m) vs. force (N) curve for each sample, an example of which is shown in Figure 3.8. The friction as estimated by the friction push from SHL 2 and the weight of the acrylic blocks used to transfer the force from the hydraulic ram to the clay specimen were subtracted from the total energy. Each remoulding test was conducted at the same or similar push rate but had a different amount of soil pushed. The total energy (J) was normalized by the volume of soil pushed (m^3) in order to remove the differences in energy values due to volume of soil extruded. The normalized total energy of each push falls within a range of 1.8E5 to 3.9E5 kJ/m^3 as seen from Figure 4.8.

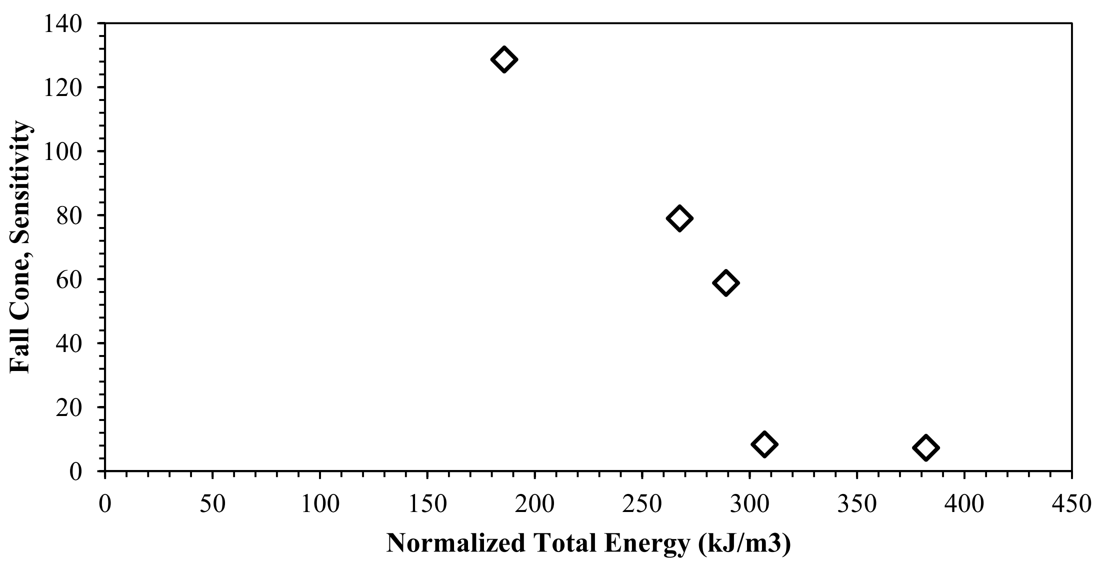


Figure 4.8: Sensitivity as determined by fall cone on intact, and remoulded specimens.

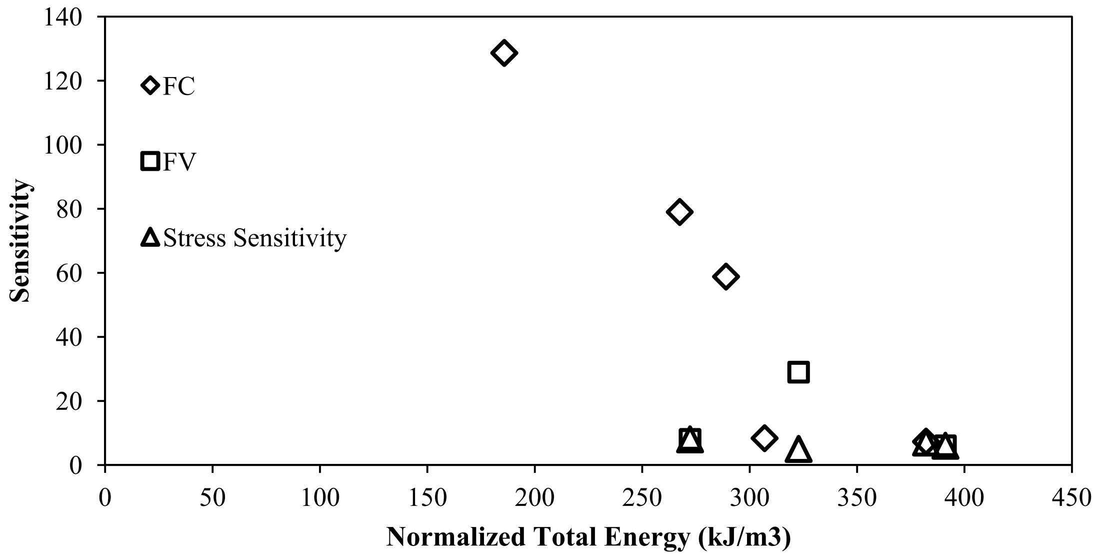


Figure 4.9: Sensitivity measured by fall cone (FC), field vane (FV), and intrinsic compressibility (Stress sensitivity) versus the normalized total energy.

Figure 4.9 shows the normalized remoulded energy plotted versus the sensitivity found using different methods. It can be noted from Figure 4.9 that there is a scatter of the results. Based on the limited results of sensitivity obtained from the fall cone, there seems to be a general trend of decrease in remoulding energy as the sensitivity increases. It should be noted, however, the sensitivity measurements are highly variable, especially when determining the remoulded strength and are generally less accurate the higher the sensitivity (Eden and Kubota, 1961). More data are needed to validate the possible trend shown in Figure 4.9.

After pushing the soil through the remoulding apparatus, some specimens were also hand remoulded for a specified amount of time in order to understand if the remoulded strength would decrease or not. The amount of time the already remoulded soil was additionally hand remoulded is denoted in the legend of Figure 4.10 and Table 4.2. Values of remoulded strength for all tests are very low and vary over a narrow range (4.2 - 0.2 kPa). Scatter in the results may be due to different cone weights used, or the small amount of water lost during a test. Soil pushed through the remoulding apparatus was tested in the fall cone about 5 minutes after remoulding; a short enough time for thixotropic processes to not affect the remoulded strength (Skempton and Northey, 1952). The water content was approximately constant, varying at the most by 2% over 30 minutes for each test on remoulded soil. It seems that most of the water content was lost from the surface of the remoulded soil while it was kept in the test cup and the values of fall cone penetration would decrease very slightly over time. The variation in remoulded strength between the strength found immediately after remoulding, and the strength found after hand remoulding is very small as the largest variation is 1.8 kPa over 45 minutes additional hand remoulding. Though there is very little difference between values of remoulded strength for either method of remoulding, even a small difference can cause a large change in the value of sensitivity. This suggests that it may be more useful to look at the remoulded strength rather than the sensitivity when assessing the mechanical behaviour of a sensitive clay.

Table 4.2: Undrained shear strength found by fall cone using clay immediately after remoulding, and after additional hand remoulding.

		Fall Cone	**Additional Minutes Hand Remoulded (kPa)**								
Sample	**Average Depth (m)**	**Remoulded Strength (kPa)**	**5**	**15**	**20**	**25**	**30**	**40**	**45**	**60**	**Cone Weights Used for Remoulded Tests**
SHL 3	20.02	0.8	0.2	-	-	-	-	-	-	-	10 g
SHL 4	10.56	3.0	-	1.6	-	-	1.2	-	1.2	1.0	60 g
	10.66	0.3	-	-	1.0	-	-	0.7	-	0.7	Initial remoulded = 60 g; Hand remoulded = 10 g
SHL 5	4.17	0.3	1.1	0.7	-	-	-	-	-	-	Initial remoulded = 60 g; Hand remoulded = 10 g
SHL 7	16.06	4.2	3.4	2.9	-	2.7	-	-	-	-	60 g

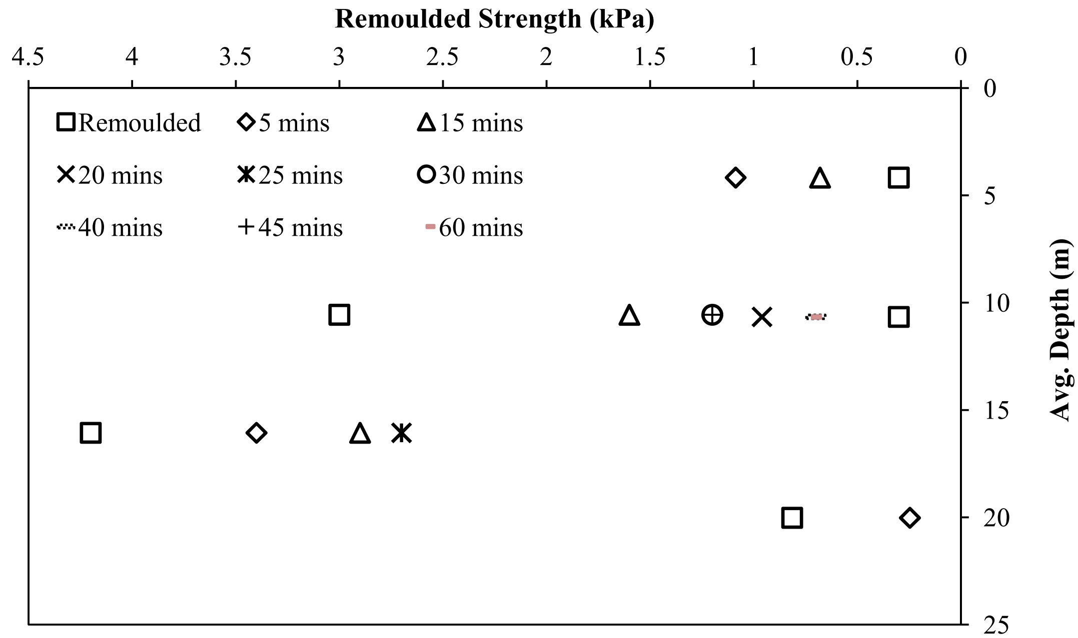


Figure 4.10: Remoulded strength of soil from the remoulding apparatus and soil that was additionally hand remoulded.

Comparing the fall cone test results to field vane results (Figure 4.11), the soil strengths measured from the fall cone were generally lower. Vertical dotted lines on Figure 4.11 show the average divides between field vane and fall cone shear strengths. It is generally known that for cohesive soils different shearing techniques result in different values of shear strength. Viscous materials, such as clay slurry, will give different values of strength based on the shear rate, and if the soil is thixotropic whether the shear rate is ascending or descending will also be a consideration (Eden and Kubota, 1961). The pore water pressure dissipation rate during a field vane test may also affect the remoulded strength. Another factor may be that the field vane tests are performed in situ while the fall cone tests are performed on shelby tube samples. The 400 g cone was used to find the undisturbed undrained shear strength in all fall cone tests.

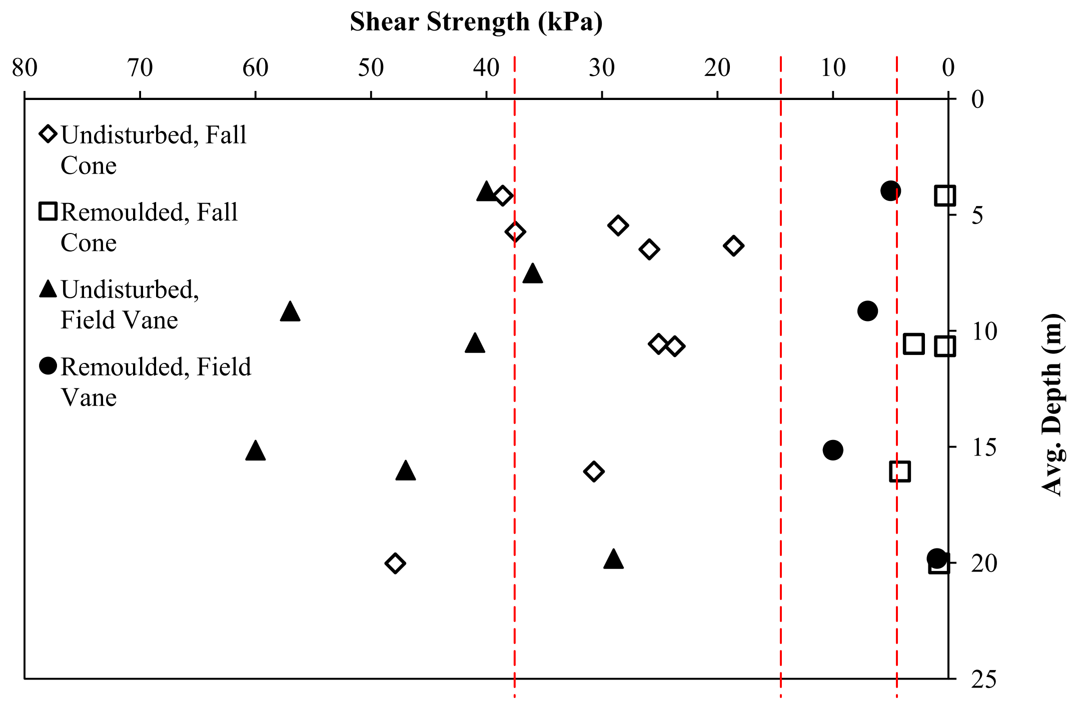


Figure 4.11: Undisturbed and remoulded strengths as measured by fall cone and field vane.

4.4 Remoulded Specimens in Cyclic Simple Shear

Four intact and four remoulded-consolidated specimens were subjected to DCSS tests at various strain amplitudes. There was an attempt to conduct the remoulded-consolidated tests at the same water content as the intact samples but it was not possible to apply the vertical pressure (39 kPa) and maintain the initial water content. The remoulded soil instead was consolidated and the water allowed to drain freely in order to simulate a boundary between clay and a granular soil when the clay has been significantly remoulded and consolidated. All tests were performed at loading frequency 1 Hz, and the results are presented in Figure 4.12 and Figure 4.13.

At cyclic shear strain amplitude of 5%, there was not good contact between the intact specimens and the bottom porous stones near the end of each test. This was because the specimen slid with the rings, instead of deforming, and a small disconnect between the rings on the bottom platen and the rings surrounding the clay could be seen. Thus, the stiffness (i.e. soil shear modulus) observed during these cycles may be lower than the actual stiffness. The remoulded-consolidated specimens (D) show markedly different behaviour compared to the intact (U) specimens. At 0.1% strain amplitude, there is little degradation in the intact specimen. The shear modulus of the remoulded-consolidated specimen actually increased slightly, most likely due to additional consolidation of the specimen associated with the cycling. As expected, the intact specimens exhibited higher strength values than the remoulded-consolidated specimens in the majority of cases as shown in Figure 4.12.

The breakdown of structure can be inferred when comparing the results of intact and remoulded-consolidated specimens for each N in Figure 4.12. The intact clay exhibited a large drop in stiffness after the first cycle while the remoulded-consolidated clay did not. The remoulded-consolidated clay actually did not exhibit a large drop in stiffness with number of cycles at any strain amplitude. The ratio of the intact and remoulded secant shear stiffness for the first, 15th, and 30th cycles are shown in Figure 4.13. The figure demonstrates the difference in percent decrease of shear modulus between the intact and remoulded-consolidated specimens. At 1% and 5% cyclic shear strains, the stiffness degradation of the intact clay was a larger percentage of its previous stiffness than the degradation of the remoulded-consolidated clay, which is shown in the drop of G_u/G_d. The stiffness of clays subjected to 5% cyclic shear strain degraded to a point where the stiffness was very low and the normal stress effectively became almost zero. At 0.1% cyclic shear strain both clays have very small changes in shear modulus with number of cycles. At 2% cyclic shear strain, the stiffness of both clays exhibited the same percentage drop with number of cycles, which is manifested in a relatively constant G_u/G_d ratio. When the G_u/G_d ratio is less than one, the disturbed specimen has a higher strength than the intact specimen. A G_u/G_d ratio that is less than one occurs at N=30 for strain amplitudes 0.1% and 5%. The higher consolidated-remoulded strength is likely because of consolidation of the slurry over the number of cycles for the lower shear strain. Slippage between the intact sample and the bottom platen at 5% shear strain amplitude may have resulted in a lower intact stiffness. The difference between the intact and consolidated-remoulded shear moduli at 5% shear strain is about 11 kPa, even though the disturbed strength is higher, it is only by 10%.

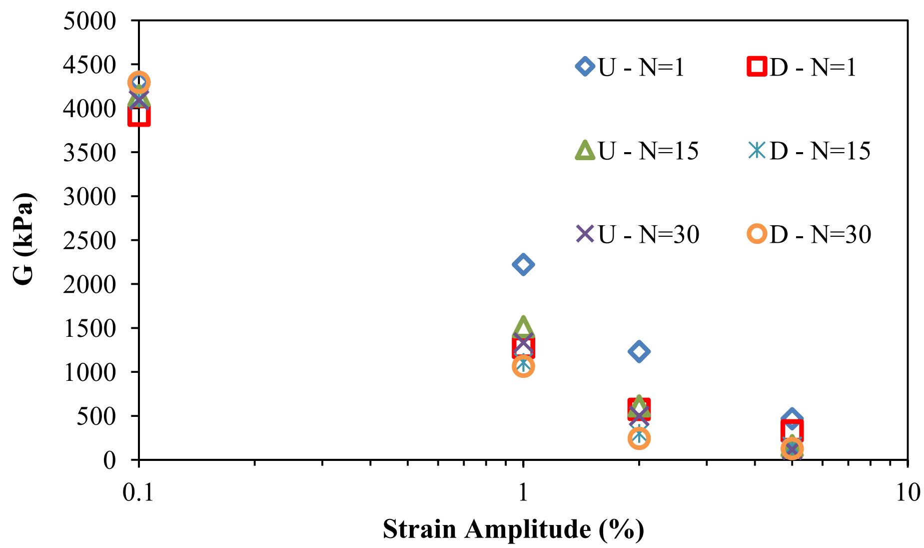


Figure 4.12: Intact and consolidated-remoulded specimens in simple shear.

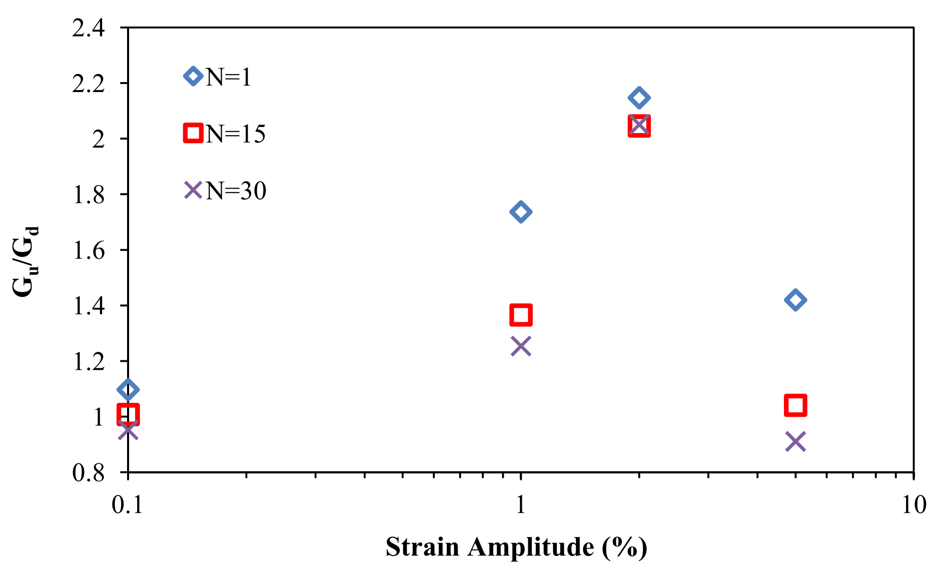


Figure 4.13: Ratio of intact shear modulus to consolidated-remoulded shear modulus change with strain amplitude.

4.5 Results of Simple Shear Tests

4.5.1 Monotonic stress-strain behaviour

Figure 4.14 displays the stress-strain behavior of undisturbed clay specimens obtained from the monotonic simple shear tests. It can be noted from Figure 4.14 that SHL 5 experienced the largest reduction in strength past the peak because it was retrieved from the weathered crust and has a yield stress ratio approximately equal to 3, which is more than double most other specimens. Other specimens also exhibited reduction in strength past peak though it was not as large as in the SHL 5 case. This behaviour is typical of sensitive clays. Failure at peak strength for all specimens happens between 2 and 4%.

In previous studies on sensitive Canadian clays using a DCSS apparatus different observations were reported: Lefebvre and Pfendler (1996) reported static peak failure had occurred between 2-3%; Silvestri et al. (1989) reported static peak failure between 1-3%; and Yong and Tang (1983) reported static peak failure between 4-5%. At large strains past peak stress, the influence of non-uniform stress on the response of the DCSS increases with strain (DeGroot et al., 1994) so that stress values at large strains do not necessarily reflect the ‘true’ soil behaviour.

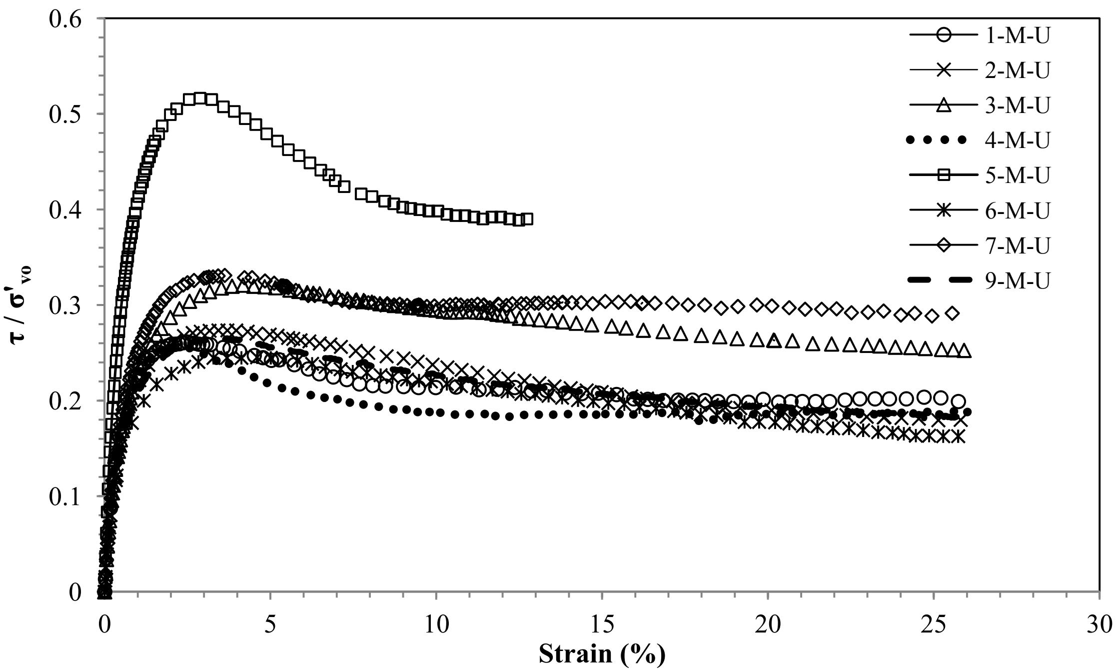


Figure 4.14: Monotonic simple shear results for all specimens.

4.6 Effect of Strain Rate

Though monotonic tests are usually conducted at low strain rates (5%/hr or less), cyclic tests are conducted at very high strain rates. The effect of strain rate on shear strength of sensitive clays in eastern Canada has been investigated by Vaid et al. (1979) and Lefebvre and LeBoeuf (1987) through triaxial tests, and Lefebvre and Pfendler (1996) through cyclic simple shear tests. They found that the shear strength increased approximately 12 - 13% per log cycle over strain rates of 0.01 to 4000 %/hr. Graham et al. (1983) investigated the effects of strain rate on the shear strength behaviour of a number of different clays, and reported that the shear strength increased linearly by 9-19% over a log cycle of 0.1 - 1%/hr.

To calculate the shear strain rate from the cyclic shear tests, the first quarter cycle max strain in percent is divided by the time to reach said strain in hours. Therefore, if a strain-controlled test has a cyclic shear strain amplitude of 1% and is performed at a frequency of 1 Hz the 1% would be divided by 6.9E-5 hrs (or 0.25 seconds) resulting in a strain rate of 14,400 %/hr. Figure 4.15 presents the range of strain rates applied in this study. The effect of strain rate on observed shear strength is clearly demonstrated, which shows that structured specimens experienced an increase in shear strength over one log cycle of approximately 12%.

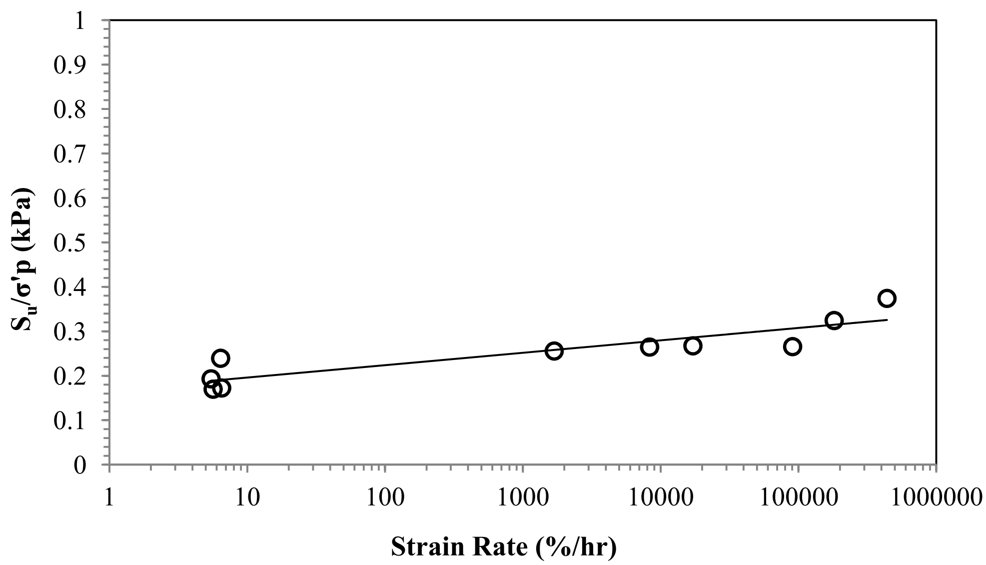


Figure 4.15: Effect of strain rate on the undrained shear strength of cyclic and monotonic specimens.

4.7 Behaviour of Clays in Cyclic Simple Shear

4.7.1 Strain-controlled tests – effect of loading frequency on shear modulus

Clay specimens were subjected to loading at frequencies of 0.1, 1, 5, and 10 Hz at shear strain amplitudes of approximately 0.1, 0.5, 1 and 5% in order to evaluate effects of frequency and strain amplitude on stiffness degradation. It can be noted from Figure 4.16 that the stiffness decreased as the strain amplitude increased for a given frequency. For the frequency 0.1 Hz, a parabolic function could fit the data well to represent the stiffness degradation with strain amplitude. However, the shear modulus degradation at other frequencies demonstrated a trend closer to a linear variation with strain amplitude. Comparing the results in Figure 4.16 a, b, c and d, it is noted that the shear modulus generally increased with increasing frequency except for the case of 10 Hz frequency. It should be noted that the upper limit of the DCSS machine employed in testing was 10 Hz and the CSR vs. horizontal strain data show that there is a small stress bias, which affected the stiffness results. All tests conducted at 10 Hz were performed on specimens from SHL 2. Heterogeneity between samples might have also been responsible for part of the scatter in results. Finally, it should be noted that the tests conducted at 5% cyclic strain amplitude could not be run at 10 Hz due to the limitations of the DCSS machine.

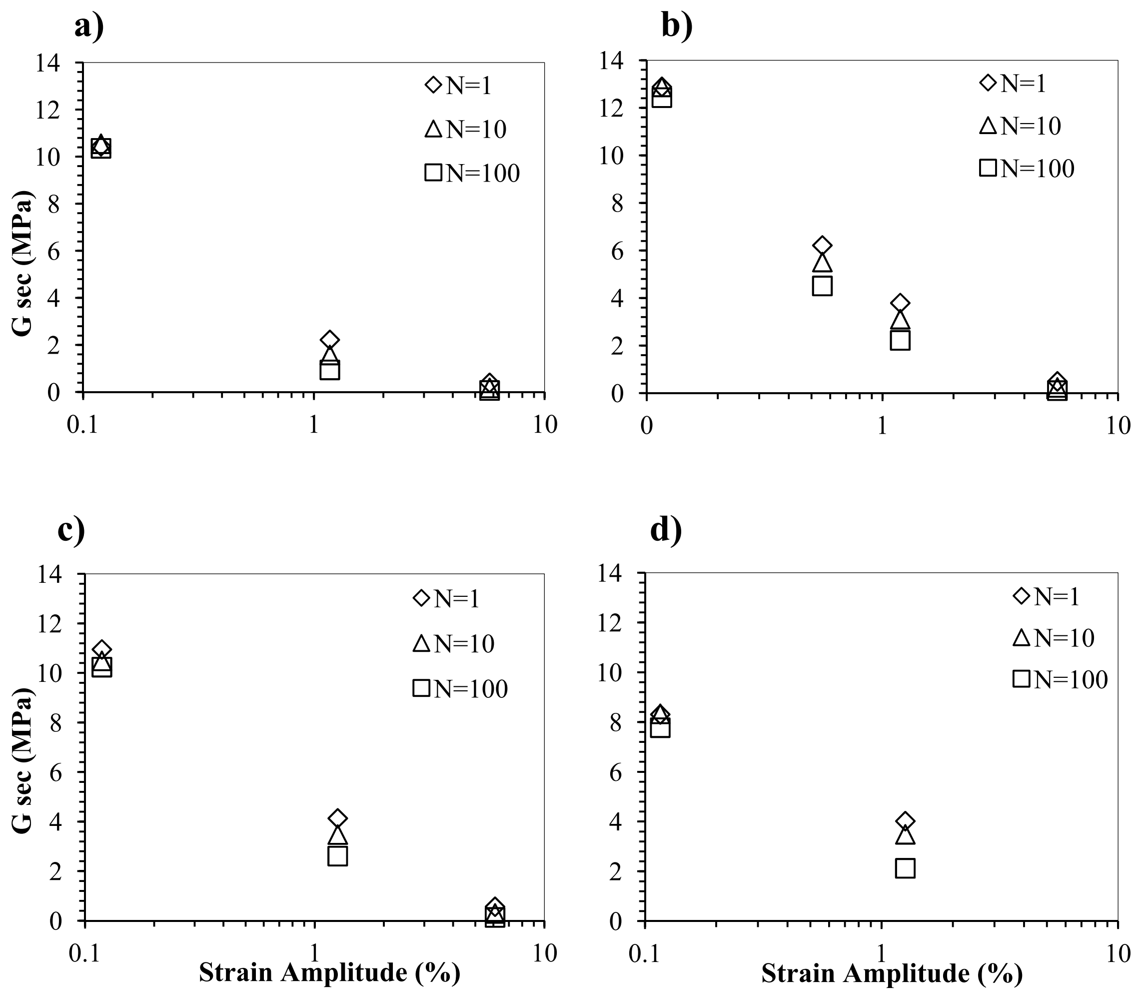


Figure 4.16: Stiffness degradation at, a) 0.1Hz b) 1 Hz c) 5 Hz d)10 Hz.

Figure 4.17 shows the change in secant shear modulus with frequency for different strain amplitudes. At 0.1% amplitude there is not much change over the range of frequencies to 5 Hz. At 10 Hz, the same issues that were pointed out earlier are evident again. If a point from the RC tests is added to the plot at 0.1% strain amplitude and N=1 is fitted with a logarithmic trendline, the shear modulus increases by 21% per log cycle. The RC point is taken from the data closest to 0.1% shear strain amplitude which is 0.132% at 28.8 Hz. Most reviewed studies had investigated the small strain behaviour of sensitive clay. One study using mixtures of Ariake clay and silica sand did do tests at various frequencies and strain amplitudes up to 1%. The Ariake 30% clay mixture had an increase of 13% per log cycle while the 100% clay mixture had an increase of 21% per log cycle at 1% shear strain amplitude. An increase of about 40% per log cycle was found for Leda clay. An increase of 6% and 9% at 0.1% strain amplitude for ACC30 and ACC100 respectively was found while it was 21% for Leda clay.

At 1% there is a constant increase in shear modulus with frequency up to 5 Hz. For any cycle number, N, the rate of increase is the same at about 1.9 MPa per log cycle. At 5% the increase in stiffness over a log cycle is 0.11 MPa for N=1. As the number of cycles increased, the slope of increase in stiffness decreased. This decrease in slope may be due to the degree of structural breakdown. At 100 cycles the specimen is significantly de-structured that the drop in shear modulus happens at a slower rate. At N=10, the increase over a log cycle was approximately 0.08 MPa. At N=100, the increase over a log cycle was 0.04 MPa.

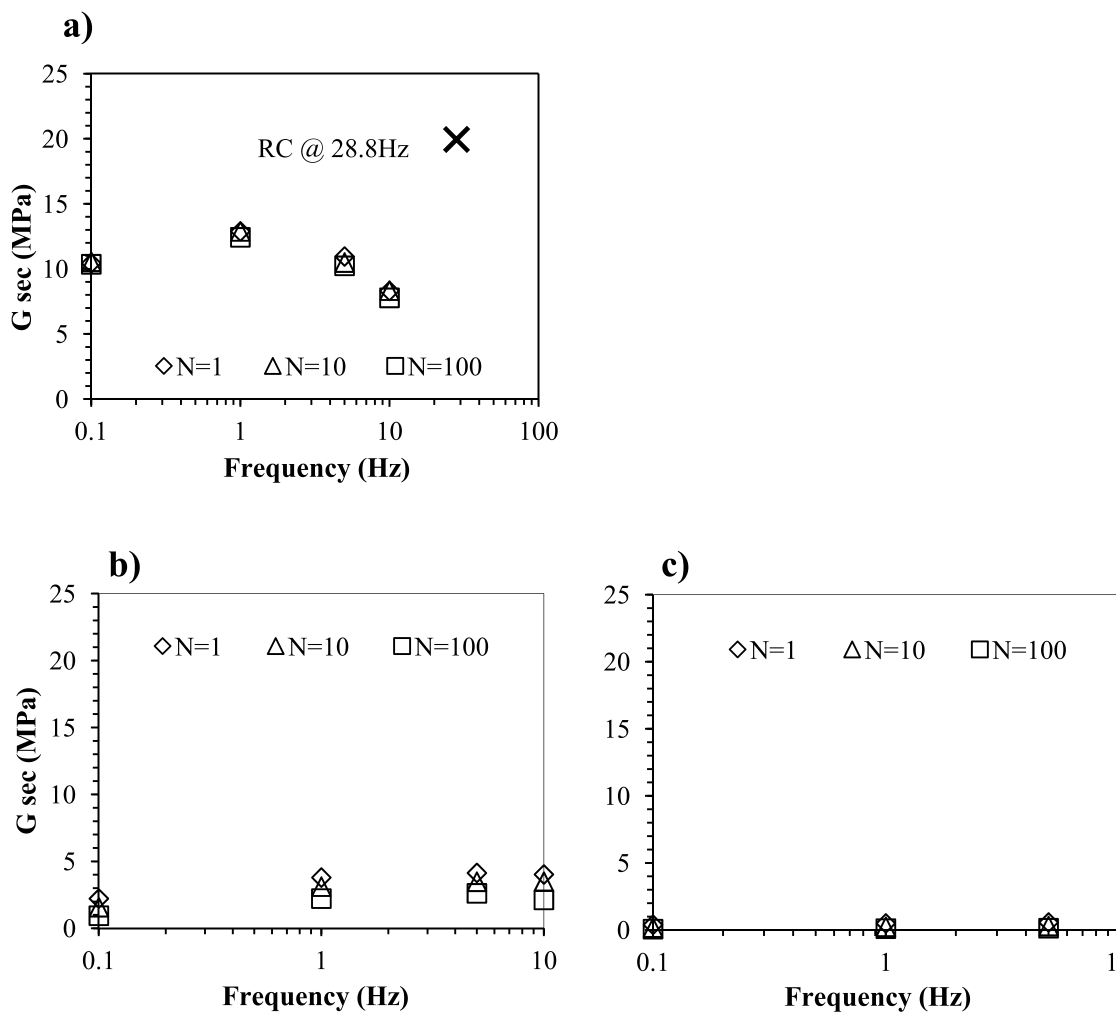


Figure 4.17: Change in shear modulus with cyclic strain amplitude of a) 0.1% b) 1% and c) 5%

4.7.2 Strain-controlled tests - evaluation of excess pore-water pressure

The variation of excess pore water pressure ratio ($\Delta u/\sigma'_v$) with the strain amplitude for each frequency tested is shown in Figure 4.18 below. The pore water pressure was taken as equivalent to the change in vertical stress (Bjerrum and Landva, 1966). As can be noted from the figures, for a shear strain amplitude 0.1% applied for all loading frequencies, very little excess pore water pressure (EPWP) was generated; the amplitude is low enough that the structure of the clay was not significantly disturbed. At 1 % strain amplitude, the EPWP decreased with increasing frequency up to 5 Hz. At frequency of 10 Hz, the EPWP increased slightly (about R_u = 0.04) compared to the observation at 5 Hz. As the frequency increased at the same strain amplitude (increasing strain rate), it has been shown by Lefebvre and Pfendler (1996) that the strength of sensitive clay increases. At strain amplitude 5%, large EPWPs are generated and the specimens fail rapidly. The first cycle shows evidence of the strain rate effect but at N=10 the specimen's structure has collapsed so far as to be very close to an R_u value of unity. At EPWP ratios close to unity, the strain rate effect is no longer evident.

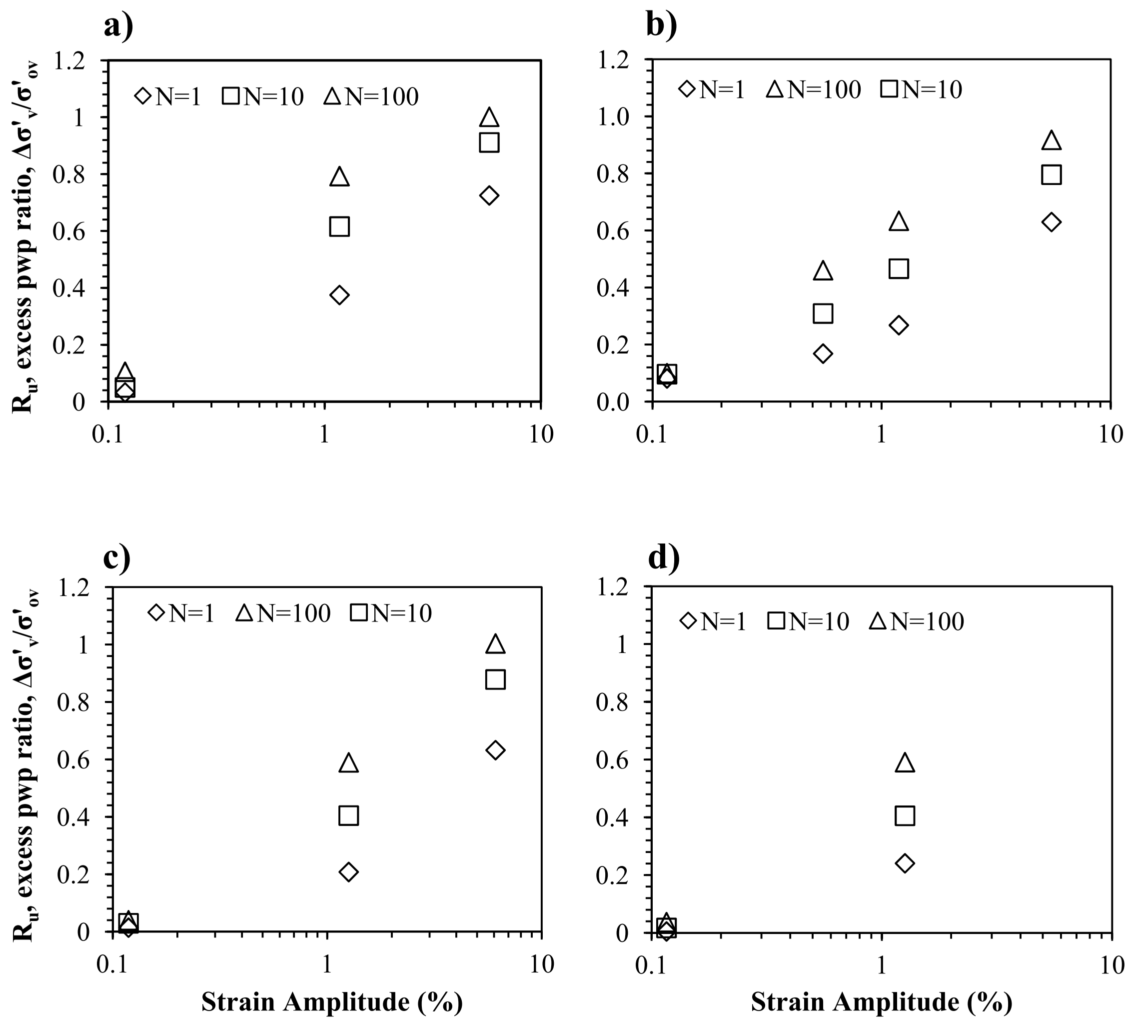
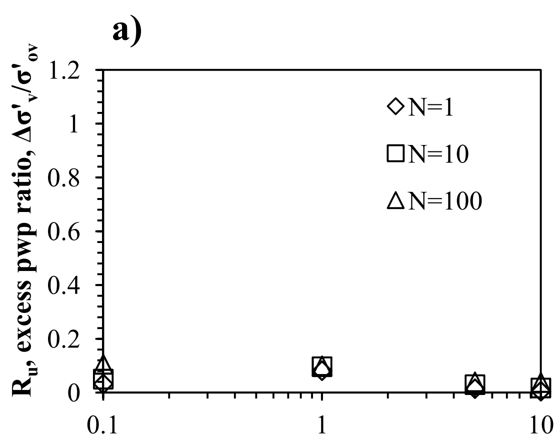


Figure 4.18: Change in R_u for, a) 0.1 Hz b) 1 Hz c) 5 Hz d) 10 Hz.

The values of excess pore water pressure generated at each strain amplitude over a range of frequencies are plotted in Figure 4.19. At 0.1% cyclic shear strain amplitude, there was not much change in the generation of EPWP over the range of frequencies investigated and the EPWP ratio did not exceed 0.12. At 1% cyclic strain amplitude there was a decrease of 0.17 in R_u with frequency for N=1 with a similar drop for other N. The decrease in slope was the same for any cycle N up to 5 Hz and corresponded to a drop of 0.135 per log cycle. A large amount of EPWP was generated after the first cycle at 5% cyclic shear strain amplitude and there was no obvious trend in EPWP over the range of frequencies. Figure 4.19 suggests that the damage done after one cycle at 5% negated the differences in EPWP generated for different frequencies. Comparing the effect of frequency and cyclic strain amplitude on the generation of EPWP, it is clear that cyclic strain amplitude has more pronounced effect than frequency. Over a range of frequencies for a single strain amplitude, the change in EPWP was less.



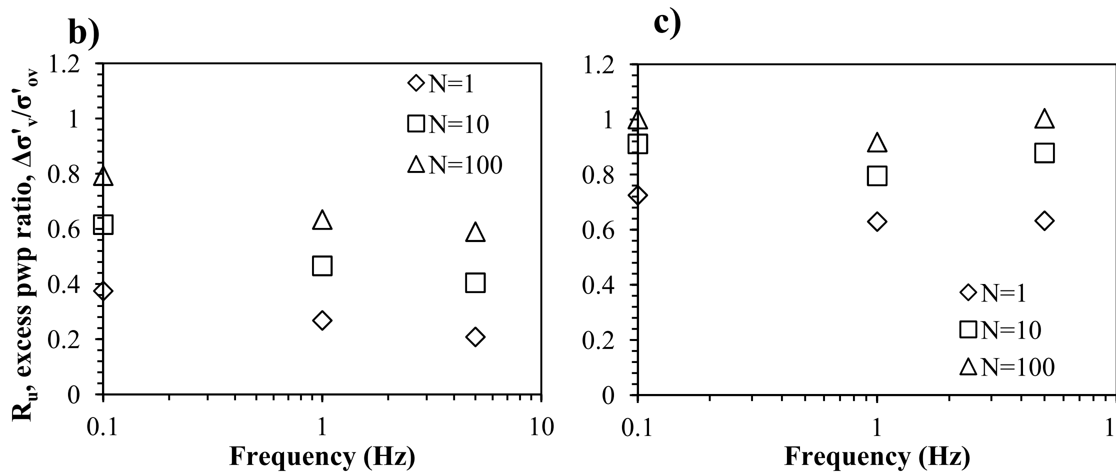


Figure 4.19: Change in R_u with cyclic strain amplitude for a) 0.1% b) 1% c) 5%

4.8 Shear Modulus Degradation Curve

A resonant column test and four DCSS tests were performed on specimens from the same soil sample and at the same confining pressure, and the results were used to establish a shear stiffness degradation curve. The normalized G/Gmax curve is plotted in Figure 4.20 and is compared to Vucetic and Dobry (1991). The PI of the clay used was evaluated employing the Cassagrande cup and was found to be 37. Vucetic and Dobry (1991) stated that sensitive clays, independent of PI, may have curves quite different than those they have plotted, which is supported by the results obtained in this study. At the same PI, the sensitive clay has much less degradation at low strain, a large drop in stiffness once a certain point is reached, and at large strains has a similar rate of degradation for N=1. The difference between the two sets of data confirms the important role that structure plays in affecting the degradation behaviour of sensitive clay.

Though the DCSS tests are strain-controlled, conventional resonant column (CRC) tests are stress-controlled and therefore different strain rates are used to determine each data point. Because the strength of clay is strain rate (or frequency) sensitive, and CRC tests are performed at the resonant frequency which is substantially higher than the frequency that the DCSS tests were performed at, the shear modulus values extrapolated to similar strain amplitudes do not agree. Khan et al. (2010) performed conventional and non-resonant RC, and cyclic triaxial (CT) tests in order to investigate the frequency effect at small strains. The authors provided a methodology to reconcile the CRC and CT shear modulus values. Applying the method of Khan et al. (2010) and using the alpha (α) value published for NR tests on a bentonite- sand mixture (12% bentonite with 32% saturation) at small strain ($\gamma < 10^{-6}$), the CRC results of the present study were adjusted. The CRC values reduced when adjusted to 1 Hz as shown in Figure 4.22. After adjustment, the normalized G/Gmax curve is plotted again against the curves from Vucetic and Dobry (1991) in Figure 4.23. The DCSS data shifted toward the proper PI line but neither the RC data not the DCSS data fit the right curve. It was not possible to perform a NR resonant column test during this study but having a Leda clay specific α would give a more accurate confirmation of how the RC results reconcile with the DCSS results.

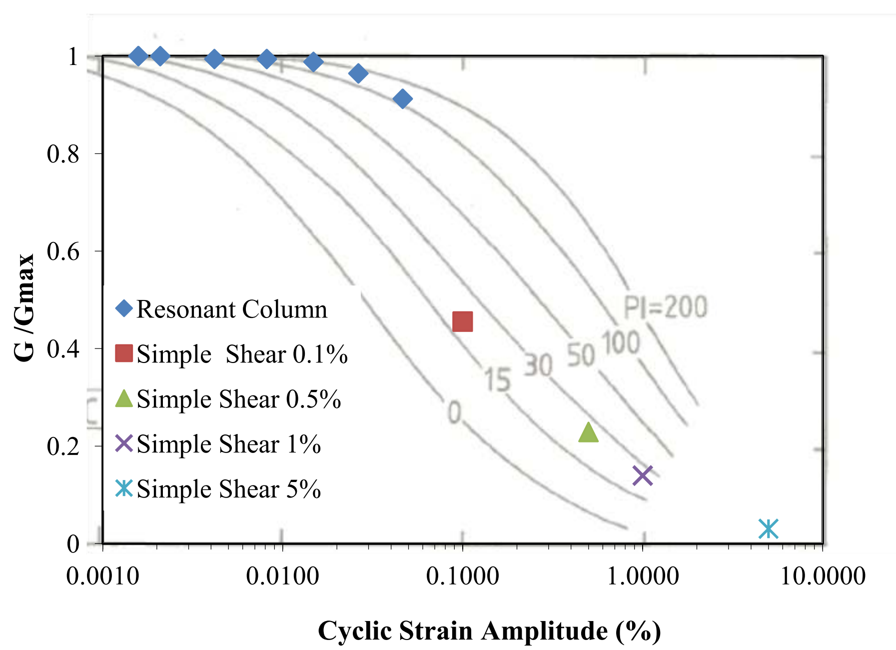


Figure 4.20: Resonant column and simple shear normalized shear modulus results at various strain amplitudes.

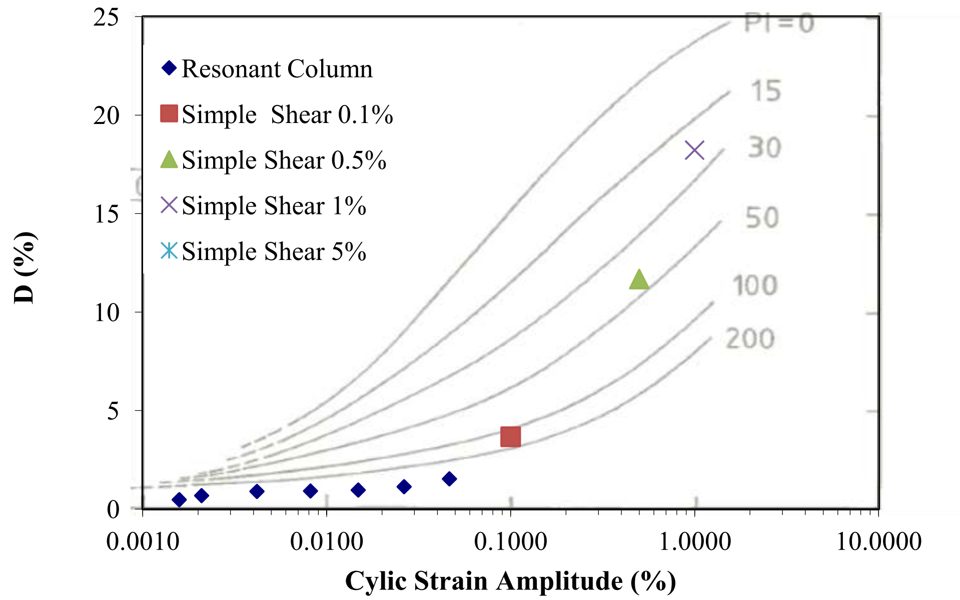


Figure 4.21: Resonant column and simple shear damping results at various strain amplitudes.

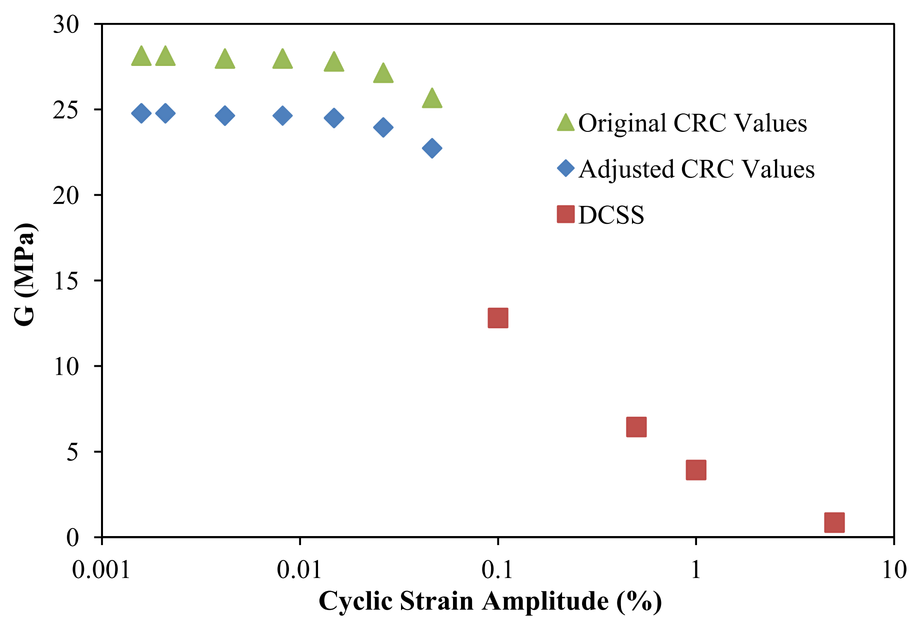


Figure 4.22: Adjusted resonant column values of shear modulus.

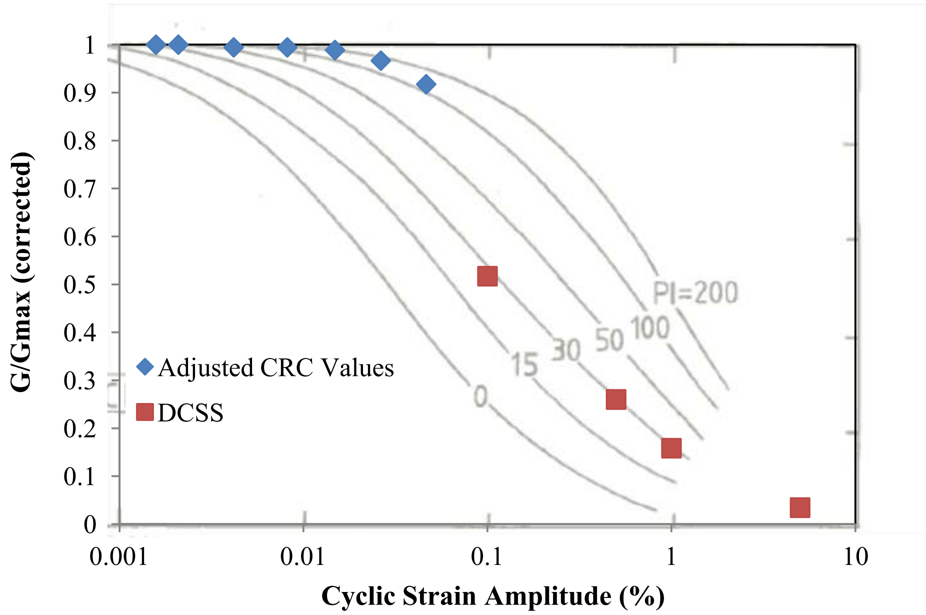


Figure 4.23: Shear modulus normalized by adjusted shear modulus for resonant column and simple shear results.

4.9 Pore-Water Pressure Threshold Cyclic Shear Strain

The threshold cyclic shear strain for pore water pressure generation, γ_t, is the value that separates the domains of negligible and permanent cyclic pore water pressure development for fully saturated soils. The clay samples tested for the estimation of γ_t were retrieved from elevations below the water table, and were tested immediately after being removed from their wax, therefore they were assumed to be saturated. Based on measurements taken during DCSS tests and the specific gravity, the saturation of both samples plotted in Figure 4.24 were greater than 97%. The data points plotted in Figure 4.24 were obtained from tests at three different frequencies and at a strain amplitude of 0.1%. The soil specimen tested at 10 Hz was cut from SHL 2, while the specimen tested at 5 Hz was obtained from SHL 3, and the specimen tested at 1 Hz was from the sample used in the RC.

Hsu and Vucetic (2006) determined γ_t by identifying the stage in which the vertical effective stress (equal to the pore water pressure in DCSS tests) does not noticeably change. They provided an example at the end of 10 cycles between stages of different cyclic shear strain: one stage had a 0.9% decrease in vertical stress while the subsequent stage produced a 3.6% decrease therefore determining that the γt can be found in the range of cyclic strain amplitudes between the two stages. Multiple stage tests at small strain amplitudes were not carried out in this study. However, the mid-cycle vertical stress at N=10 and 0.1% cyclic strain amplitude were used to determine an informed estimate of the pore-water pressure threshold shear strain for the two samples shown. The test conducted at 10 Hz, had a 0.9% change in vertical stress at the end of 10 cycles and so γ_t was estimated to be approximately between 0.1 and 0.12%. The test conducted at 5 Hz had a 2.7% decrease in vertical stress after 10 cycles. Because the decrease is noticeable at 2.7%, the results were extrapolated back and the threshold shear strain was estimated to be between 0.02 and 0.04%. At loading frequency 1 Hz, the decrease in vertical stress after 10 cycles was 1.4%, which translates to γ_t approximately 0.1 to 0.07%. The loading frequency is not known to have an effect on γ_t (Hsu and Vucetic, 2006) and the results observed in this study seem to confirm this; there is no noticeable variation with frequency in Figure 4.24. Though this study can only provide estimates of γ_t for various samples of Leda clay, it seems as if the effect of PI on the cyclic pore-water pressure threshold shear strain was predicted well by the framework provided in Hsu and Vucetic (2006).

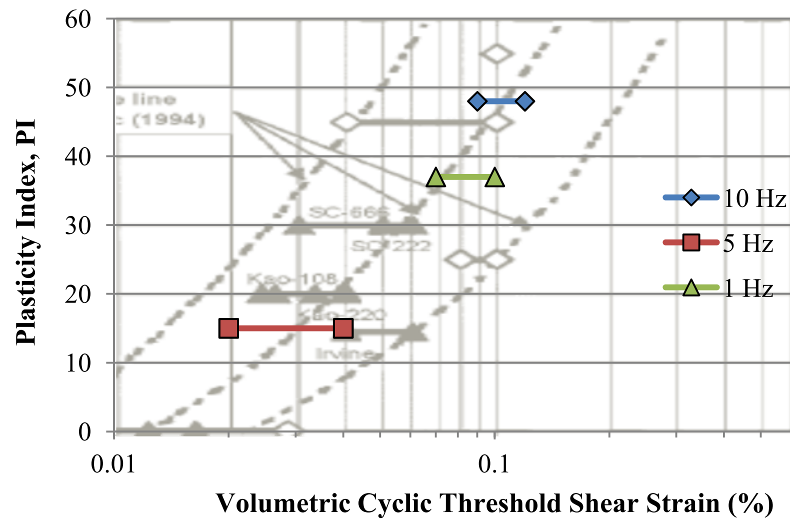


Figure 4.24: Volumetric threshold shear strain range for Leda clay specimens.

4.10 Stress-Controlled Cyclic Simple Shear Tests

Most geotechnical earthquake engineering design is carried out using the results of stress-controlled tests at various cyclic stress ratios. Stress-controlled tests were carried out at the vertical in situ stress and at the yield stress to determine the influence of structure at different yield stress ratios (YSRs) on the cyclic strength of Leda Clay. Figure 4.25 a) and b) show the progression of failure at each cyclic strength ratio (CSR) for the YSR at 1 (de-structured) and at 2 (structured). The failure of the specimens was marked by a steep increase in strain with the number of cycles.

It is common to plot the CSR at 3% strain vs. N for clays (Boulanger and Idriss, 2004). Figure 4.26 shows the CSR for both YSR values at the number of cycles to +/- 3%. DCSS tests on St. Alban clay (Lefebvre and Pfendler, 1996) at an YSR of 2.2 were adjusted from 0.1 Hz to 1 Hz (Boulanger and Idriss, 2004) and are also shown in Figure 4.26 for comparison. Data from the YSR=2 tests compare well with the data on the St. Alban clay. The specimens tested at YSR=1 were consolidated to 100 kPa and were allowed to drain. A power trend line through YSR=1 has a 'b' value of 0.153 and 'a' value of 1.85, which are both higher than the 'b' value of 0.129 and 'a' value of 1.426 for

St. Alban clay. Therefore, consolidating the clay at or past its yield stress would increase the cyclic strength of the clay at lower N but the clay at YSR=1 degrades at a faster rate and fails more sharply than the clay at YSR=2.

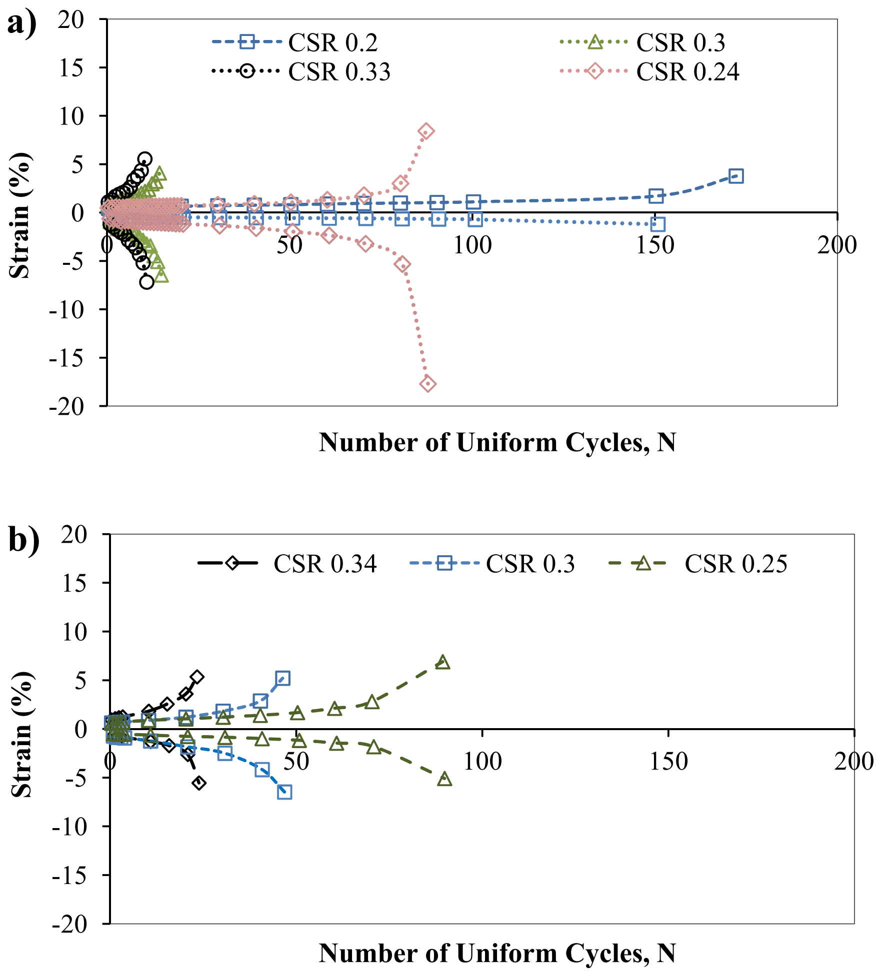


Figure 4.25: Increase in strain with number of cycles for a) a yield stress ratio equal to 1 b) a yield stress ratio equal to 2

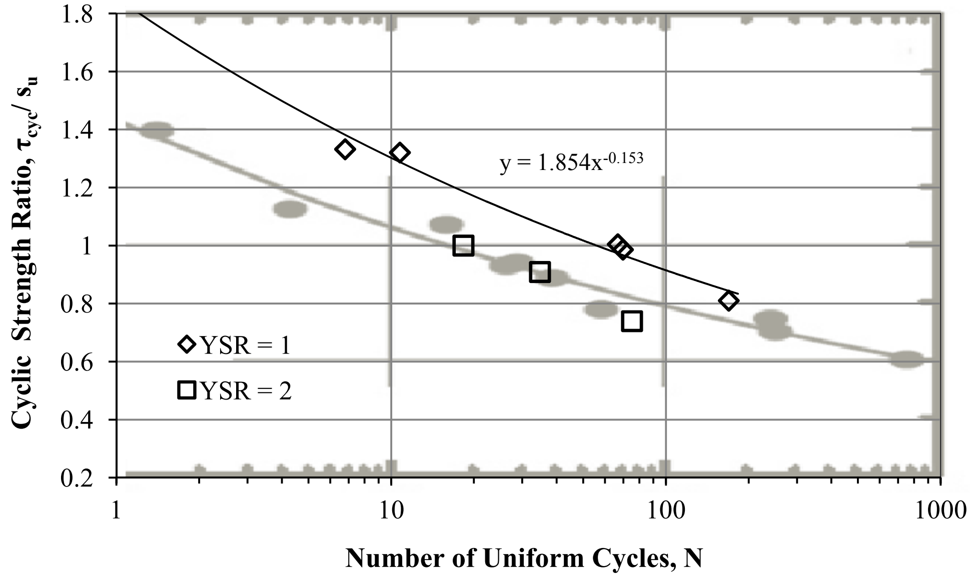


Figure 4.26: Stress-controlled tests results for Leda clay compared to those for St. Alban clay (Lefebvre and Pfendler, 1996).

Chapter 5

5 Conclusions and Recommendations

5.1 Summary

In this study, a review of the literature concluded that more data on the dynamic behaviour of Leda clay was needed. In addition, it was clear from literature survey that the energy used in remoulding sensitive clay specimens can have a significant impact on the measured sensitivity. An apparatus that is able to measure energy during remoulding is developed and employed to produce remoulded specimens of sensitive clay. Oedometer tests are used to determine the compressibility behaviour of both remoulded and undisturbed specimens. Static and cyclic simple shear tests are carried out in order to determine their undrained strength behaviour. In addition, a resonant column test is conducted on a sensitive clay specimen in order to provide data on its small strain properties.

A review of soil remoulding methods found that there was not a method that both allowed the remoulded soil to be used in all tests, and could quantify the energy required to remould the soil. A method in which clay was extruded though a perforated disk and wire screen while quantifying the energy used was developed. The efficiency of remoulding by the proposed method in this study was checked by hand remoulding the same soil and using the fall cone to find the remoulded strength. The remoulded strength decreased very slightly but because of the very low strengths even a small change resulted in a large change in sensitivity. It was also found that field vane tests generally give higher values of shear strength than fall cone tests. Plotting sensitivity values evaluated from fall cone test, field vane test, and stress sensitivity versus normalized total remoulding energy resulted in a scattered trend toward decreasing remoulded energy with increasing sensitivity. This may be a legitimate trend but it would need to be confirmed with additional data.

5.2 Conclusions

The large strain behaviour of Leda clay under various excitation frequencies and strain amplitudes was investigated by conducting strain-controlled tests and the effect of yield stress ratio was investigated by performing stress-controlled tests. The following observations and conclusions are offered:

1. Monotonic tests performed at the in-situ vertical pressure with a strain rate of 5%/hr demonstrated that the specimens failed within 2-4% strain.
2. Strain-controlled cyclic tests were performed with strain amplitudes of approximately 0.1%, 1%, and 5%. The clay response exhibited steep degradation of stiffness with strain amplitude for different frequencies (0.1, 1, 5 and 10 Hz). The increase in shear modulus between 0.1 Hz and 1 Hz at 1% strain amplitude is almost double the value of shear modulus at 0.1 Hz, though the rate of increase decreased with increasing frequency from 1 Hz.
3. The clay shear modulus determined at 0.1% strain amplitude increased by up to 21% over one log cycle, including the resonant column data as well. The frequency effect was also apparent at 1% and 5% strain amplitude.
4. At 5% strain amplitude, there was very little remaining strength to the clay. The cyclic strain amplitude has a more pronounced effect than frequency does on the generation of excess pore-water pressures within the clay.
5. Stress-controlled tests found that at a yield stress ratio (YSR) equal to 2, the data matched existing results for St. Alban clay performed at a similar YSR (YSR=2.2). Once the specimens were consolidated to an YSR=1, the clay actually increased in strength but also degraded at a faster rate.
6. Adjusting the shear modulus values obtained from the resonant column tests and those obtained from the DCSS for the frequency effect (using the method in Khan et al. (2010)) resulted in a better match between the resonant column and DCSS data. It should be noted, however, that an α value should be found for Leda clay to further improve the match.
7. Comparing the shear modulus degradation curves of Leda clay with the Vucetic and Dobry’s (1991) clay degradation curves at different plasticity indices, which

demonstrated that Leda clay does not behave in a similar way to non-sensitive clay. However, adjusting the shear modulus values obtained from the resonant column test for the frequency effect yielded a better fit for the DCSS test results to Vucetic and Dobry's curves but not for the resonant column results.

8. The results of pore-water pressure threshold cyclic shear strain for three samples fit well into the range of values found by Hsu and Vucetic (2006) suggesting that this approach is applicable and useful for engineers dealing with Leda clay.

Cyclic simple shear and oedometer tests were also performed on remoulded and consolidated specimens and then compared to intact specimens subject to the same loading conditions. Based on the results from this test series, the following conclusions may be drawn:

1. The intact specimens had a higher initial stiffness than remoulded-consolidated sensitive clay specimens.
2. Larger stiffness degradation was observed for the intact specimens between N=1 and N=15 at the same strain amplitude, relative to that of reconstituted specimens.
3. The sensitivity framework was used to find the sensitivity of three different samples of Leda clay. It was found that the values of stress sensitivity matched well with sensitivity values found by field vane, which suggests that the sensitivity framework can be applied to Leda clay.

5.3 Recommendations for future work

It is recommended to perform more strain-controlled and stress-controlled cyclic tests in order to clarify the trends observed in this study, for clays with different sensitivity values. For a more precise measurement of the threshold shear strain, tests such as those described in Hsu and Vucetic (2006) should be undertaken. The trend of increasing sensitivity with decreasing normalized total energy of remoulding would need to be clarified as well with more testing.

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Appendix A - Test Reports

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